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Light Gage Steel Design Manual

American Iron and Steel Institute

LIGHT GAGE STEEL DESIGN MANUAL

JANUARY, 1949

MERICAN IRON AND STEEL INSTITUTE
50 FIFTH AVENUE, NEW YORK 1, N. Y.

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DESIGN MANUAL

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PRICE ONE DOLLAR

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350 FIFTH AVENUE, NEW YORK 1, N. Y.

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LIST OF SYMBOLS

A = Area in square inches
a = Spacing of attachments of wall material to stud, measured along length of stud, in inches
B = Length of bearing, in inches, except in Tables 1 to 6
b = Effective design width, see Sec. 2.2, Page 27
b/t = Ratio of effective design width to thickness of an element
D = Diameter
D = Amount of curling in per cent of depth
d = Depth of lip of stiffened element
E = Modulus of elasticity
f = Unit stress
f_b = Basic design stress
f_c = Safe or allowable compressive stress
f'_c = Actual compressive stress
f_y = Unit stress at yield point
h = Depth of beam, in inches, except as provided in Sec. 3.4 on Page 33
I = Moment of Inertia
K = A factor which is the ratio between the actual allowable working stress in an unstiffened element and the basic design stress. See Footnotes of Table 2, Page 16 and Table 5, Page 22.
k = A factor (modulus of elastic support); see Sec. 5, Page 38
L = Length of span
M = Bending moment
M = Limiting load factor (or constant) used in determining effective design width; Sec. 2.3.1, Page 28
N = Limiting deflection factor (or constant) used in determining effective design width; See Sec. 2.3.1, Page 28
P = Force or concentrated load
Q = A factor used in column design, See Sec. 3.6.1, Page 35
r = Radius of gyration;
S = Section modulus;
S′ = Section modulus based upon reduced cross section having effective design width
t = Thickness in inches
v = Unit shear stress
W = Total load
w = Flat width of an element
w/t = Ratio of flat width to thickness of an element
x = Distance parallel to X axis
$\bar{x}$ = Distance to centroid parallel to X axis
y = Distance parallel to Y axis
$\bar{y}$ = Distance to centroid parallel to Y axis

LIGHT GAGE STEEL DESIGN MANUAL

A Manual of Basic Sections and Design Data

for

Light Gage Cold Formed Steel Structural Members

PART I

FOREWORD

The use of light gage structural sections, made by cold-forming sheet or strip steel, is increasing in popularity with designers. High-speed manufacturing methods have been developed, adding to the economy of production. The fire safety gained by the use of an incombustible type of construction also has strong appeal.

Light gage sections frequently are specified for various light-load structural purposes, in which members lighter than the conventional hot-rolled shapes can be used to advantage. These applications include residential, industrial, commercial, educational and miscellaneous structures.

In April, 1946, the American Iron and Steel Institute published its "Specification for the Design of Light Gage Steel Structural Members." This Manual of Basic Sections and Design Data is intended to supplement the Design Specification and to facilitate its application to ordinary design problems. Properties and other design information with respect to a useful series of basic sections are included in tabular form, as follows:

2 Channels with Stiffened Flanges, Back-to-Back
2 Channels with Unstiffened Flanges, Back-to-Back
2 Equal Leg Unstiffened Angles, Back-to-Back
Single Channels and Zees with Stiffened Flanges
Single Channels and Zees with Unstiffened Flanges
Equal Leg Unstiffened Angles

The properties of these sections are tabulated for steel sheet or strip of the following thicknesses:

Thickness Inches	Nominal Gage Number
.135	10
.105	12
.075	14
.060	16
.048	18

Designers are by no means limited to the use of sections listed in this Manual. The flexibility of the forming processes and the great variety of shapes which may be formed of sheet and strip steel are such that substantial economies can sometimes be effected, or particular use requirements met, by the use of special sections. However, the designer should seek the advice of manufacturers or fabricators before specifying any special sections.

SCOPE

The structural properties and other data given herein will be helpful in applying the Design Specification to determine the safe load capacities and deflections of shapes made of any combination of flat elements formed from sheet or strip steel less than 3/16 in. thick.

The provisions of the Design Specification and the data contained in this Manual will give accurate results for different grades of carbon and low alloy steels, and may be applied without substantial error to sections made of stainless steel, but *they do not apply to non-ferrous metals.*

EXPLANATORY COMMENTS

Fundamental design procedures which are universally applied in the selection of hot-rolled steel shapes are equally applicable in dealing with light gage cold-formed sections. However, one of the distinguishing characteristics of the latter sections is that the compression elements are usually relatively wide and thin compared to the compression elements normally encountered in heavier structural shapes. Because of this, some modification of conventional design procedure is necessary in computing the structural properties of light gage cold-formed sections.

After they have been properly determined, the application of the properties (section modulus, moment of inertia, radius of gyration, etc.) to the computation of safe loads and deflections is the same as for the heavier shapes. The fact that in some cases properties are not constant for a given section, but vary with the unit stress, complicates the procedure slightly, but basic design principles and practices remain the same.

The necessity for modifying conventional procedures in the computation of properties of light gage sections arises from the fact that wide, thin compression elements tend to buckle at unit stresses which are lower than the yield point of the material and which vary with the ratio of width to thickness of the element. The Design Specification recognizes two types of compression elements, as follows:

> *Unstiffened Elements,* flat elements which have one unstiffened edge parallel to the direction of stress; such as the compression flange of a plain channel or I-section.
>
> *Stiffened Elements,* those in which both edges parallel to the direction of stress are stiffened by connection to a web, flange, stiffening lip or other similar elements; such as the top flange of an inverted U-section, or the lipped flange of a channel or I-section. See Chart No. 1, p. 5.

(For proportioning of channels and zees for maximum effectiveness, see pages 42, 43.)

Reduced Working Stresses

In computing the properties of light, formed sections, no modification of conventional procedures is made in dealing with *unstiffened* compression elements, but where the ratio of width to thickness of such elements is greater than 12, the basic design stress, f_b, is decreased to a lower safe or allowable working stress, f_c, as the ratio of width to thickness increases. (See Reduced Properties, p. 3.)

Reduced Cross Section

In the case of *stiffened* compression elements in which the ratio of width to thickness does not exceed 25, conventional design procedures apply without modification; but in computing the properties of sections having stiffened compression elements with a ratio of width to thickness greater than 25, only a part of any such element is used; the remaining portion is not considered effective for load-carrying purposes, and there is no reduction in the basic design stress.

Variable Properties

The width of stiffened elements which is considered effective varies not only with the ratio of width to thickness of the element but also with the unit stress in the element. Since the actual unit stress in a flexural member is a function of the bending moment, the effective width, as well as the moment of inertia, varies with the bending moment. In determinations where the actual unit stress is unknown, such as deflection calculations, the moment of inertia, therefore, is related to the particular bending moment under consideration; for here the actual unit stress (or bending moment) governs, rather than the basic design stress which is used in load determinations. Accordingly, in some of the Tables of Properties in Part II of this Manual, the moments of inertia to be used in deflection calculations appear as variable quantities, expressed in terms of the bending moment. The application of the variable properties, given in the tables, to the computation of deflections is precisely the same, however, as in conventional design procedure where the properties are constants.

Reduced Properties

A number of short-cuts and mechanical variations from the above procedure have been found convenient and are embodied in some of the data given in the Tables of Properties in Part II. For instance, it is frequently simpler, in the case of sections having unstiffened compression elements, to work with a constant basic unit design stress (such as 18,000 p.s.i.) than with a variable unit stress, the precise value of which depends upon the width to thickness ratio of the compression element. This can be done simply by introducing into the Tables of Properties a factor, "K", which is the ratio between the actual allowable working stress in the unstiffened element and the basic design stress. By multiplying the section modulus of the section by K, a reduced section modulus is obtained which can be used with the full basic design stress to give correspondingly safe load values. It is this reduced section modulus which has been tabulated for such unstiffened sections.

Compression Members

In the case of compression members (columns and struts) the procedure for considering the reduced strength of unstiffened elements and the reduced effective width of stiffened elements has been simplified by the introduction of another factor, "Q". (See Chart No. 2, p. 6, and Sec. 3.6.1, Design Specification, p. 34.)

The important point for the designer to bear in mind, however, is that all of these factors and apparent departures from usual practice are really only mechanical devices intended to facilitate the application of variable stresses to unstiffened elements and variable effective widths to stiffened elements.

DESIGN CHARTS 3A to 3D — EFFECTIVE DESIGN WIDTHS

Charts 3A to 3D give the effective design width ratio (b/t) for various flat width ratios (w/t) and unit stresses. They may be used for any grade of steel at any of the plotted or interpolated unit stresses. Charts 3A and 3B are to be used in computing properties for safe load determinations and Charts 3C and 3D are to be used in computing properties for deflection determinations.

The different values shown for the two different determinations are explained by the fact that the effective design width varies inversely with the unit stress, becoming less as the unit stress increases. Consequently, in order to develop a yield-strength safety factor of 1.85 (corresponding to that specified for basic design stresses) the effective width for load determinations must be that corresponding to a load 1.85 times the actual safe load. The effective width, "b", for safe load determination, therefore, is not that corresponding to the actual stress under the safe load, but to the unit stress under 1.85 times that safe load. Hence, the "b" values, based upon the curve values of Charts 3A and 3B, are those corresponding to unit design stresses 1.85 times those designated in the charts.

In determining deflection, however, the "b" value corresponding to the *actual* unit stress under the applied load must be used. (The safety factor does not enter these determinations.)

Charts 3A to 3D are based on the equations for b/t as given in Sec. 2.3.1, Design Specification, p. 28. Charts 3B and 3D give the b/t values somewhat more clearly than Charts 3A and 3C for low ratios of w/t. In Charts 3B and 3D, all curves below the dot and dash line give the b/t values when w/t is equal to or less than *M* or *N* respectively; all curves above the dot and dash line give the b/t values when the w/t is greater than *M* or *N* respectively, where *M* and *N* are the criteria for change of formulas, as defined in Sec. 2.3.1, Design Specification, p. 28.

How to Use the Charts

Determine the w/t value of the section being investigated. Enter the chart with the w/t value just determined, follow to the allowable unit stress, and read the corresponding b/t value.

See Examples in Part V.

DESIGN CHART 4 — COLUMN DESIGN CURVES

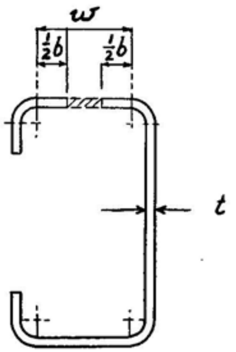
Chart 4 of this Manual may be used in connection with the tabular values of "Q", in the selection of compression members. Since "Q" may be considered a stress factor as well as a shape or buckling factor, the curves of Chart 4 include values of Q which are greater than unity, in order that they may be used for grades of steel other than Grade C. The procedure for applying Chart 4 to other grades of steel is noted on the chart.

See Examples in Part V.

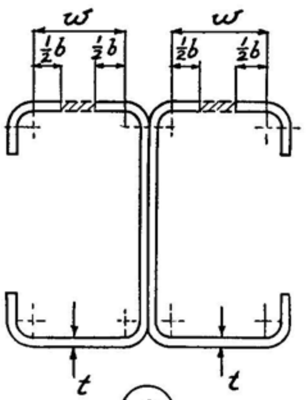
(See Section 3.6.1, Design Specification)

EFFECTIVE CROSS SECTIONS
OF MEMBERS IN BENDING

CHART No. 1



1



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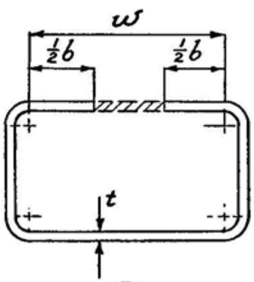

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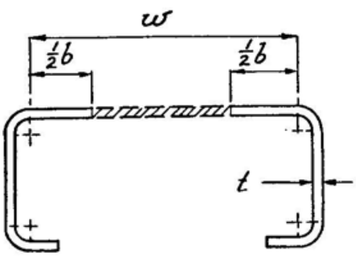

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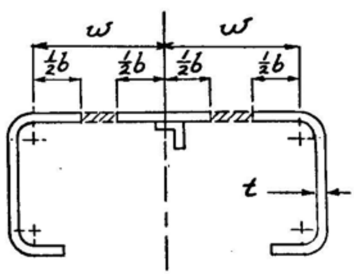
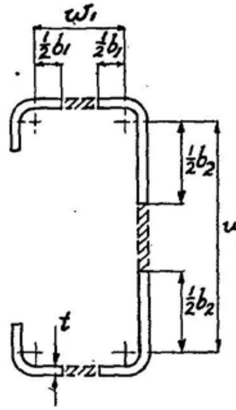


5

EFFECTIVE DESIGN AREA FOR DETERMINING "Q" FOR CROSS SECTIONS OF MEMBERS IN COMPRESSION

(See Section 3.6.1)

$$Q = \frac{\text{Total Area} - \text{Shaded Area}}{\text{Total Area}} \left(\begin{array}{l}\text{Where All Elements Are} \\ \text{Stiffened At Both Edges}\end{array}\right)$$



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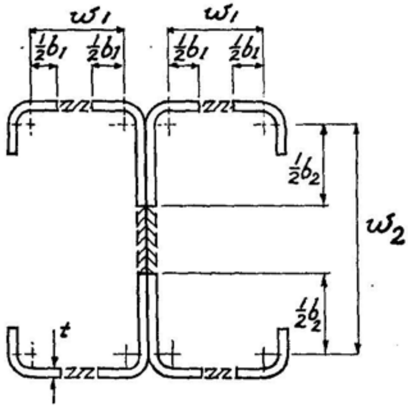

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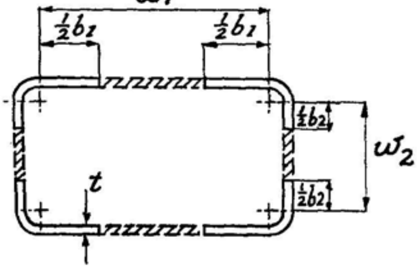
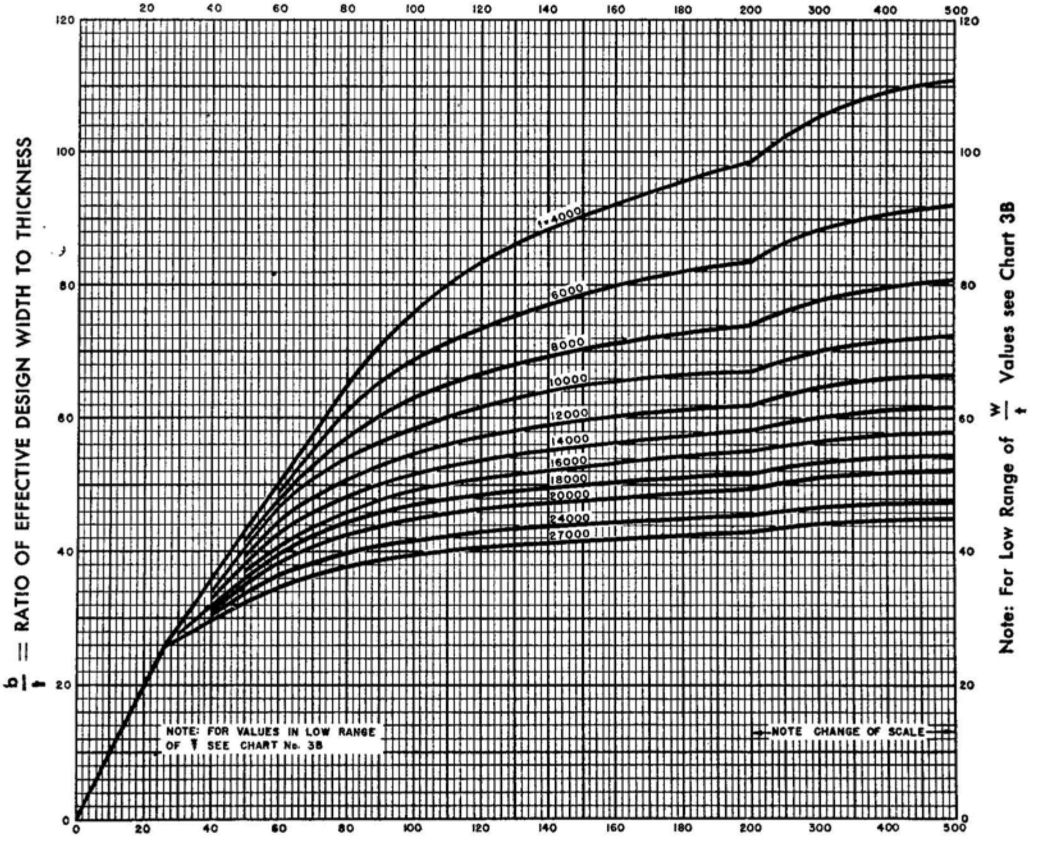


8

EFFECTIVE DESIGN WIDTH
FOR
LOAD DETERMINATION ONLY

COMPRESSION ELEMENTS STIFFENED ALONG BOTH EDGES
(High Ratios of w/t)



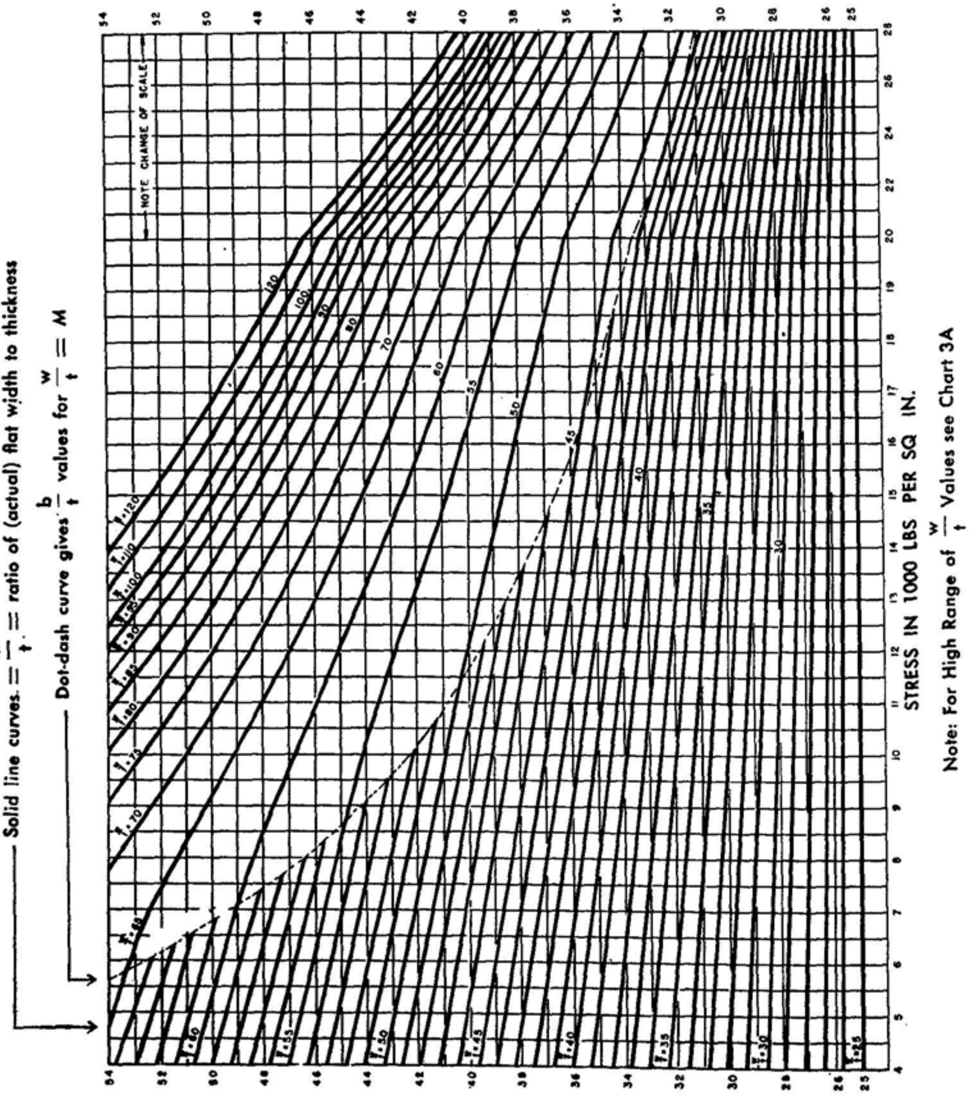
$\frac{w}{t}$ = RATIO OF (ACTUAL) FLAT WIDTH TO THICKNESS

EFFECTIVE DESIGN WIDTH FOR LOAD DETERMINATION ONLY

CHART No. 3B

COMPRESSION ELEMENTS STIFFENED ALONG BOTH EDGES
(Low Ratios of w/t)

$\frac{b}{t}$ = RATIO OF EFFECTIVE DESIGN WIDTH TO THICKNESS



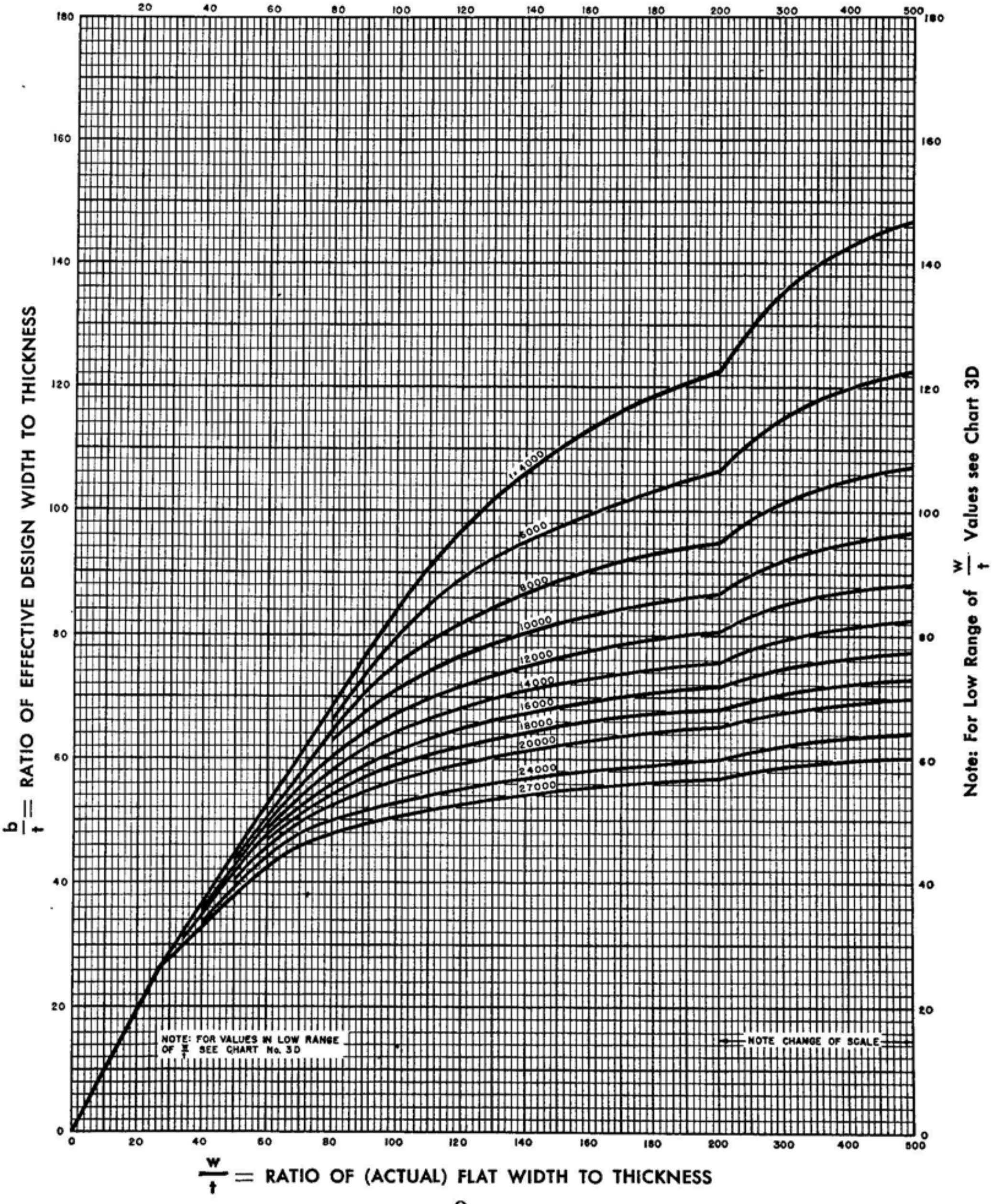
$\frac{b}{t}$ = RATIO OF EFFECTIVE DESIGN WIDTH TO THICKNESS

EFFECTIVE DESIGN WIDTH FOR DEFLECTION DETERMINATION ONLY

CHART No. 3C

COMPRESSION ELEMENTS STIFFENED ALONG BOTH EDGES

(High Ratios of w/t)

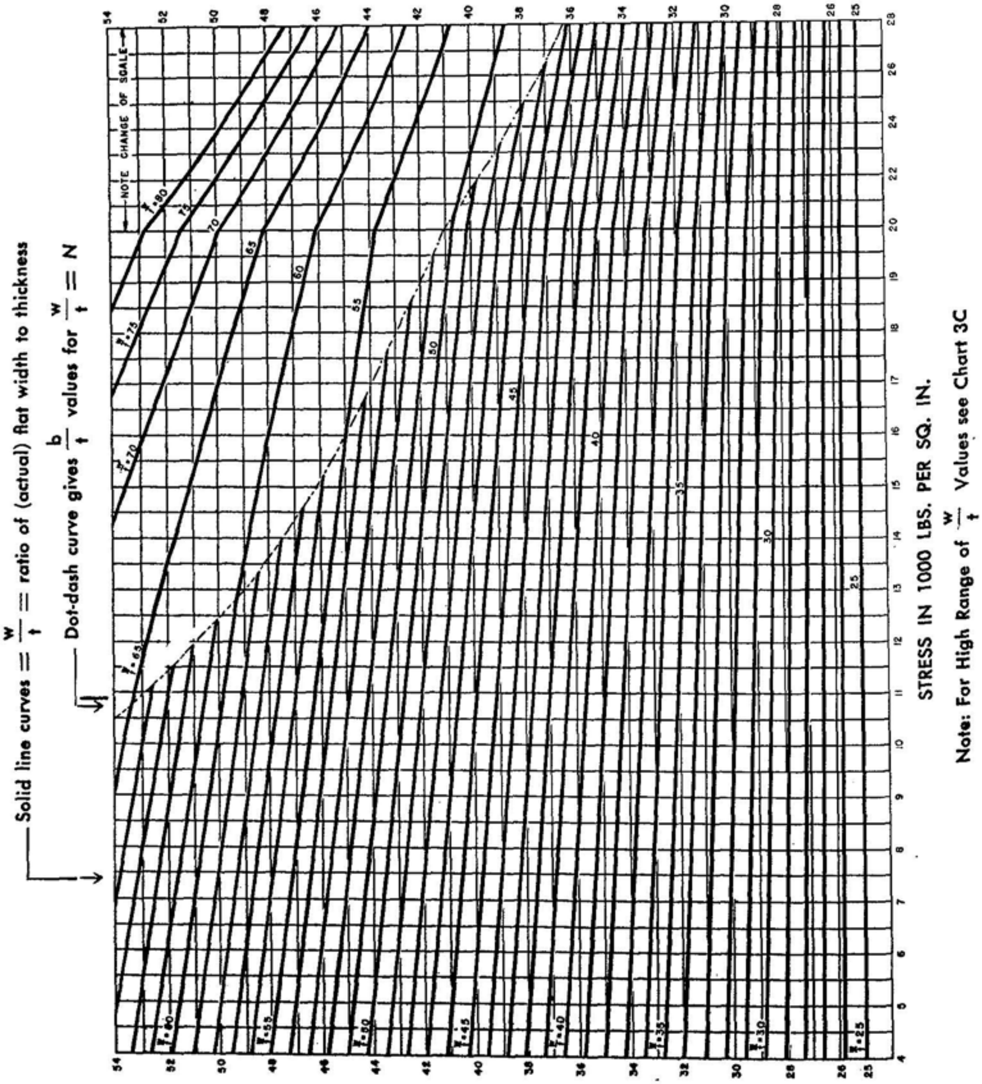


EFFECTIVE DESIGN WIDTH
FOR
DEFLECTION DETERMINATION ONLY

CHART No. 3D

COMPRESSION ELEMENTS STIFFENED ALONG BOTH EDGES
(Low Ratios of w/t)

$\frac{b}{t}$ = RATIO OF EFFECTIVE DESIGN WIDTH TO THICKNESS



$\frac{b}{t}$ = RATIO OF EFFECTIVE DESIGN WIDTH TO THICKNESS

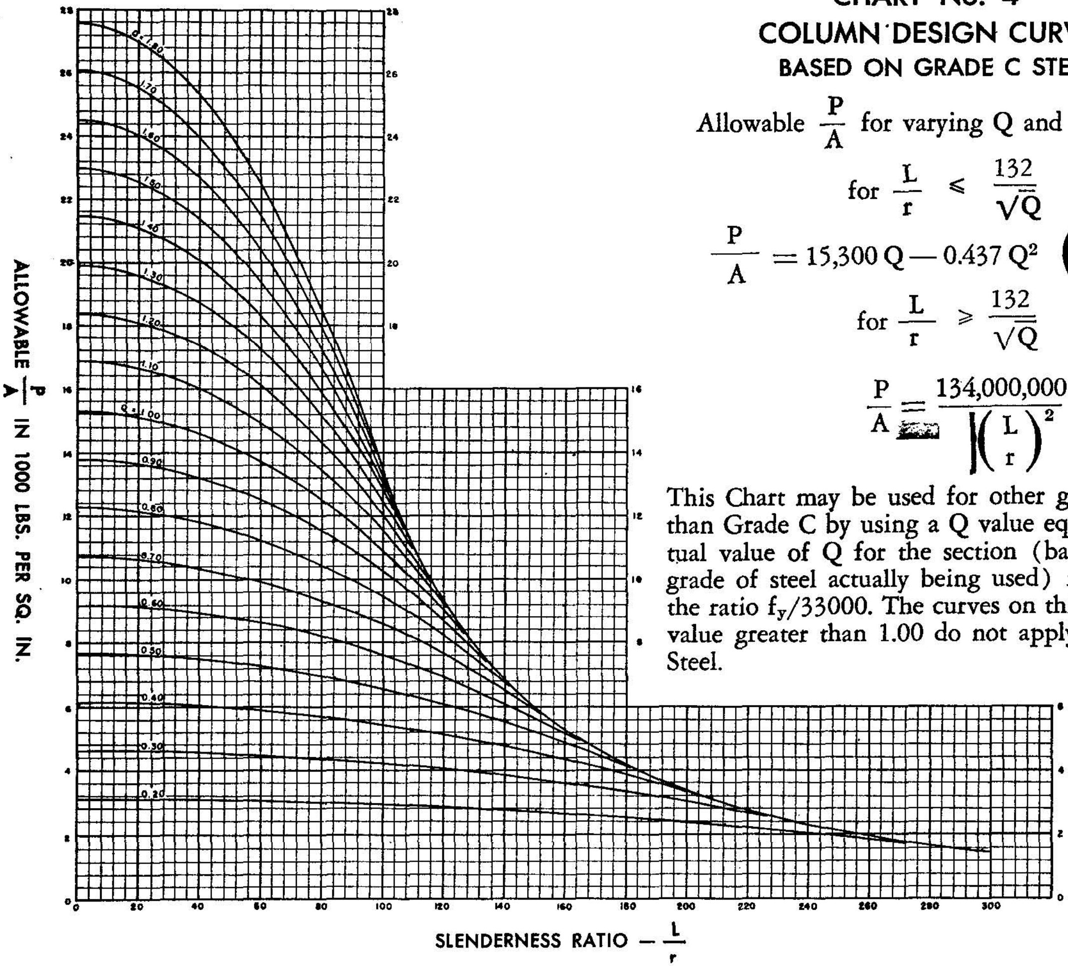


CHART No. 4
COLUMN DESIGN CURVES
BASED ON GRADE C STEEL

Allowable $\frac{P}{A}$ for varying Q and $\frac{L}{r}$ values

for $\frac{L}{r} \leqslant \frac{132}{\sqrt{Q}}$

$$\frac{P}{A} = 15{,}300\,Q - 0.437\,Q^2 \left(\frac{L}{r}\right)^2;$$

for $\frac{L}{r} \geqslant \frac{132}{\sqrt{Q}}$

$$\frac{P}{A} = \frac{134{,}000{,}000}{\left(\frac{L}{r}\right)^2}$$

This Chart may be used for other grades of steel than Grade C by using a Q value equal to the actual value of Q for the section (based upon the grade of steel actually being used) multiplied by the ratio $f_y/33000$. The curves on this chart for Q value greater than 1.00 do not apply to Grade C Steel.

PART II

EXPLANATION OF TABLES OF PROPERTIES

The tables which follow give approximate weights, also dimensions and design properties for a selected group of sections. In these tables:

f_b is the basic design stress,

S_x and S_y are section moduli about the X-X and Y-Y axes respectively,

I_x and I_y are moments of inertia of the section about the X-X and Y-Y axes respectively.

In Tables 1 and 4, which pertain to sections having stiffened compression flanges, the moment of inertia to be used in deflection computations, for some of the sections, varies with the unit stress and hence with the bending moment. For any particular case where the bending moment M is known, the value of I_x can be obtained by inserting the known value of M into the expressions given in the tables.

r_x and r_y are radii of gyration about the X-X and Y-Y axes respectively. These are based on the full theoretical outline of the section as specified in Sec. 3.6, Design Specification, p. 34.

Q is the column factor defined on p. 34, Sec. 3.6.1, Design Specification.

The properties in the tables which follow have been tabulated both for Grade C steel (33,000 pounds/sq. in. specified minimum yield point, $f_b =$ 18,000 pounds/sq. in.) and for a high strength steel having a specified minimum yield point of 50,000 p.s.i. and $f_b = 27{,}000$ p.s.i.

Interpolation and Extrapolation

Properties which vary with the basic design stress f_b, may be found for other grades of steel by straight line interpolation and extrapolation provided f_b is not less than 13,500 p.s.i. nor more than 30,000 p.s.i. The procedure for such determination may be expressed by the following formula:

$$F_f = F_{18} + \left[(F_{27} - F_{18}) \frac{f_b - 18000}{27{,}000 - 18{,}000} \right], \text{ in which}$$

F_f is the property desired.

F_{18} and F_{27} are the corresponding properties for $f_b = 18{,}000$ and $f_b = 27{,}000$ p.s.i. respectively.

f_b is the basic allowable design stress for which the interpolated property F_f is desired.

In using the above formula careful attention must be given to the sign of each quantity shown.

The following properties are subject to straight line interpolation in the manner outlined above.

Table	Properties
1	S_x, Q_{min}, Q_{max}
2	S'_x, S'_y, K, Q_{min}, Q_{max}
3	f_c, M_{max}, Q
4	S_x, Q
5	S'_x, K, Q
6	f_c, M_{max}, Q

The properties given in Tables 1 to 6 inclusive are, in general, properties of appropriately reduced sections and are directly applicable to ordinary design computations. (See Reduced Cross Section, p. 3, also Reduced Properties, p. 3.)

Full section properties, unreduced for either effective width of stiffened elements or decreased working stresses on unstiffened elements, are given in tabular form in the Appendix. These unreduced properties should not be used in direct design computations; for such use the reduced properties given in Tables 1 to 6 are mandatory. However, the unreduced properties, along with the other information given in the tables of the Appendix, may be found useful in less frequent design investigations, particularly in connection with built-up members.

Beams Having Large h/t Ratios

For single web beams of high strength steel with large h/t ratios, the allowable fiber stress should not exceed f_b, nor should it exceed:

$$f_{max} = 18{,}000\left(\frac{170}{h/t}\right)^2$$

Shear stresses in these beams shall be investigated according to Sec. 3.4 of the Design Specification.

Channel and Zee Sections

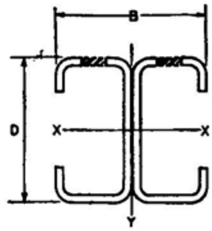
Single channel and zee-sections are simple economical shapes which can be manufactured without welding or other joining in contrast to I-shaped sections, obtained by joining two channels back-to-back. Hot-rolled structural channels and zees have been satisfactorily used for many years. However, a word of warning is in order regarding the use of cold-formed light-gage channels and zees as flexural members. Both single channels and zees tend to twist if loaded in the plane of the web. In the case of channels, this is caused by the fact that the shear center of the section is not in the plane of the web. In the case of zees, the web axis does not represent a principal axis of the section; the section tends to move sideways, and since the ends are positioned against lateral movement, torsion results. Hot-rolled sections, being usually heavier than cold-formed sections and having, in addition to increased thickness, a substantial core of metal at the junction of flange and web, usually have a torsional resistance which is sufficiently high to permit them to serve satisfactorily as beams in most normal applications. The torsional resistance of thin cold-formed sections is usually very low, however; consequently thin cold-formed channels and zees should be used as beams only when they are sufficiently braced against twisting at a number of points along the span. Such bracing can usually be accomplished by reasonably closely spaced attachments to flooring and ceiling structures or by cross bracing.

Axially Loaded Compression Members

In the design and selection of axially-loaded compression members such as columns, two principles apply: reduced stress for unstiffened compression members and reduced effective design width for stiffened elements. A "Q" factor has been introduced into the Design Specification, which takes into account both of these principles, in a relatively simple manner, as is explained in Sec. 3.6.1 of that Specification, p. 34. For any particular section, the value of this Q factor is a constant when used at any particular unit stress, but changes at different stress.

TABLE I

2 CHANNELS
WITH
STIFFENED FLANGES
BACK-TO-BACK

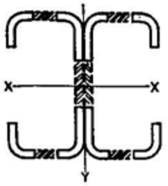


These properties do not apply unless compression flanges are adequately braced laterally.

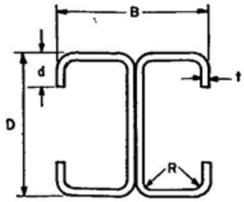
Nominal Size	Ga.	Thickness t	BEAM STRENGTH $f_b = 18{,}000$ psi S_x	S_y	BEAM STRENGTH $f_b = 27{,}000$ psi S_x	S_y	DEFLECTION Any Grade of Steel I_x	I_y
In.	No.	In.	In.^{3}	In.^{3}	In.^{3}	In.^{3}	In.^{4}	In.^{4}
D x B 12 x 7	10	.135	18.7	3.58	18.7	3.58	112.	12.52
	12	.105	14.3	2.70	14.2	2.70	87.6 −.00000243 M	9.45
	14	.075	9.56	1.81	9.28	1.81	62.8 −.00001174 M	6.33
10 x 7	10	.135	14.6	3.58	14.6	3.58	73.0	12.52
	12	.105	11.2	2.70	11.1	2.70	57.0 −.00000216 M	9.45
	14	.075	7.42	1.81	7.18	1.81	41.0 −.00001050 M	6.33
	16	.060	5.76	1.46	5.50	1.46	33.0 −.00001502 M	5.13
9 x 6½	10	.135	12.1	3.17	12.1	3.17	54.3	10.31
	12	.105	9.28	2.26	9.28	2.26	42.0 −.00000029 M	7.34
	14	.075	6.24	1.60	6.06	1.60	30.6 −.00000812 M	5.19
	16	.060	4.78	1.22	4.58	1.22	24.4 −.00001204 M	3.96
8 x 6	10	.135	9.66	2.63	9.66	2.63	38.6	7.90
	12	.105	7.56	1.98	7.56	1.98	30.2	5.94
	14	.075	5.14	1.40	5.02	1.40	22.0 −.00000588 M	4.20
	16	.060	3.94	1.06	3.80	1.06	17.6 −.00000940 M	3.19
7 x 5½	10	.135	7.50	2.14	7.50	2.14	26.2	5.90
	12	.105	5.96	1.71	5.96	1.71	20.9	4.72
	14	.075	4.16	1.21	4.08	1.21	15.3 −.00000378 M	3.33
	16	.060	3.16	0.919	3.06	0.919	12.2 −.00000704 M	2.53
6 x 5	10	.135	5.62	1.71	5.62	1.71	16.8	4.26
	12	.105	4.48	1.37	4.48	1.37	13.4	3.42
	14	.075	3.26	1.04	3.22	1.04	10.1 −.00000188 M	2.60
	16	.060	2.48	0.785	2.42	0.785	8.02 −.00000484 M	1.96
5 x 4	10	.135	3.76	1.18	3.76	1.18	9.38	2.36
	12	.105	3.02	0.950	3.02	0.950	7.52	1.90
	14	.075	2.24	0.671	2.24	0.671	5.60	1.34
	16	.060	1.74	0.500	1.72	0.500	4.46 −.00000131 M	1.00
	18	.048	1.34	0.406	1.31	0.406	3.60 −.00000369 M	0.813
4 x 4	10	.135	2.76	1.18	2.76	1.18	5.52	2.35
	12	.105	2.22	0.950	2.22	0.950	4.44	1.90
	14	.075	1.66	0.670	1.66	0.670	3.32	1.34
	16	.060	1.29	0.500	1.28	0.500	2.66 −.00000112 M	1.00
	18	.048	0.998	0.406	0.974	0.406	2.16 −.00000316 M	0.812
3½ x 4	10	.135	2.30	1.18	2.30	1.18	4.02	2.35
	12	.105	1.85	0.950	1.85	0.950	3.24	1.90
	14	.075	1.40	0.670	1.40	0.670	2.44	1.34
	16	.060	1.09	0.500	0.950	0.500	1.96 −.00000102 M	1.00
	18	.048	0.838	0.406	0.816	0.406	1.59 −.00000288 M	0.812
3 x 3½	12	.105	1.36	0.767	1.36	0.767	2.04	1.34
	14	.075	1.02	0.491	1.02	0.491	1.53	0.860
	16	.060	0.832	0.402	0.832	0.402	1.25	0.703
	18	.048	0.632	0.294	0.622	0.294	0.996 −.00000143 M	0.515

The properties of Table I apply to flexural computations only when the channels are adequately joined and when the compression flanges are adequately braced laterally.

TABLE I



2 CHANNELS
WITH STIFFENED FLANGES
BACK-TO-BACK

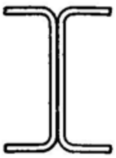


COLUMN PROPERTIES							DIMENSIONS						
Full Theoretical Outline			$f_b = 18{,}000$ psi		$f_b = 27{,}000$ psi		Depth	Flange	Lip	T'k'ness	Radius	Wt. per Foot	Nominal Size
Area	r_x	r_y	Q		Q		D	B	d	t	R		
Sq. In.	In.	In.	min.	max.	min.	max.	In.	In.	In.	In.	In.	Lb.	In.
5.40	4.56	1.52	.737	.943	.693	.924	12.0	7.0	1.0	.135	3/16	18.8	D x B
4.20	4.57	1.50	.665	.904	.621	.876	12.0	7.0	0.9	.105	3/16	14.6	12 x 7
2.98	4.59	1.46	.539	.808	.487	.764	12.0	7.0	0.7	.075	3/32	10.4	
4.86	3.87	1.60	.800	.964	.757	.952	10.0	7.0	1.0	.135	3/16	17.0	
3.78	3.89	1.58	.728	.926	.683	.903	10.0	7.0	0.9	.105	3/16	13.1	10 x 7
2.68	3.91	1.54	.594	.832	.538	.787	10.0	7.0	0.7	.075	3/32	9.34	
2.16	3.92	1.54	.530	.780	.473	.744	10.0	7.0	0.7	.060	3/32	7.50	
4.46	3.49	1.52	.829	.976	.787	.968	9.0	6.5	1.0	.135	3/16	15.5	
3.42	3.50	1.47	.768	.949	.726	.933	9.0	6.5	0.8	.105	3/16	11.9	9 x 6½
2.46	3.53	1.45	.629	.855	.574	.813	9.0	6.5	0.7	.075	3/32	8.56	
1.95	3.53	1.42	.559	.802	.501	.752	9.0	6.5	0.6	.060	3/32	6.80	
4.00	3.11	1.41	.857	.990	.817	.987	8.0	6.0	0.9	.135	3/16	13.9	
3.10	3.12	1.38	.803	.963	.762	.952	8.0	6.0	0.8	.105	3/16	10.8	8 x 6
2.24	3.15	1.37	.670	.879	.616	.842	8.0	6.0	0.7	.075	3/32	7.78	
1.77	3.15	1.34	.598	.828	.540	.780	8.0	6.0	0.6	.060	3/32	6.16	
3.54	2.72	1.29	.886	1.000	.850	1.000	7.0	5.5	0.8	.135	3/16	12.3	
2.78	2.74	1.30	.839	.978	.799	.971	7.0	5.5	0.8	.105	3/16	9.72	7 x 5½
2.00	2.76	1.29	.718	.906	.667	.876	7.0	5.5	0.7	.075	3/32	7.00	
1.59	2.77	1.26	.641	.848	.584	.803	7.0	5.5	0.6	.060	3/32	5.54	
3.08	2.34	1.18	.913	1.000	.886	1.000	6.0	5.0	0.7	.135	3/16	10.7	
2.42	2.35	1.19	.875	.996	.838	.995	6.0	5.0	0.7	.105	3/16	8.46	6 x 5
1.78	2.38	1.21	.775	.936	.729	.915	6.0	5.0	0.7	.075	3/32	6.20	
1.41	2.39	1.18	.694	.884	.639	.847	6.0	5.0	0.6	.060	3/32	4.92	
2.54	1.92	0.962	.948	1.000	.932	1.000	5.0	4.0	0.7	.135	3/16	8.86	
2.00	1.94	0.973	.907	1.000	.877	1.000	5.0	4.0	0.7	.105	3/16	7.00	
1.45	1.96	0.961	.836	.976	.796	.969	5.0	4.0	0.6	.075	3/32	5.06	5 x 4
1.15	1.97	0.934	.766	.936	.721	.915	5.0	4.0	0.5	.060	3/32	4.00	
0.922	1.98	0.939	.689	.885	.636	.849	5.0	4.0	0.5	.048	3/32	3.22	
2.28	1.56	1.02	1.000	1.000	1.000	1.000	4.0	4.0	0.7	.135	3/16	7.92	
1.80	1.57	1.03	.954	1.000	.939	1.000	4.0	4.0	0.7	.105	3/16	6.28	
1.30	1.60	1.02	.896	1.000	.863	1.000	4.0	4.0	0.6	.075	3/32	4.54	4 x 4
1.03	1.61	0.987	.832	.957	.789	.944	4.0	4.0	0.5	.060	3/32	3.58	
0.826	1.62	0.992	.754	.902	.700	.871	4.0	4.0	0.5	.048	3/32	2.88	
2.14	1.37	1.05	1.000	1.000	1.000	1.000	3.5	4.0	0.7	.135	3/16	7.46	
1.69	1.38	1.06	.982	1.000	.976	1.000	3.5	4.0	0.7	.105	3/16	5.90	
1.23	1.41	1.05	.921	1.000	.896	1.000	3.5	4.0	0.6	.075	3/32	4.28	3½ x 4
0.966	1.42	1.02	.866	.970	.826	.961	3.5	4.0	0.5	.060	3/32	3.36	
0.778	1.43	1.02	.789	.911	.736	.883	3.5	4.0	0.5	.048	3/32	2.72	
1.48	1.17	0.951	1.000	1.000	1.000	1.000	3.0	3.5	0.7	.105	3/16	5.18	
1.05	1.21	0.906	.944	1.000	.926	1.000	3.0	3.5	0.5	.075	3/32	3.64	3 x 3½
0.846	1.22	0.912	.916	1.000	.889	1.000	3.0	3.5	0.5	.060	3/32	2.94	
0.662	1.23	0.881	.843	.950	.798	.934	3.0	3.5	0.4	.048	3/32	2.32	

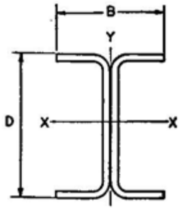
Q is the column factor (Sec. 3.6.1, Design Specification, p. 34). When the web-to-web connections are sufficiently closely spaced to cause the two webs to act as one, Q_{max} applies. When such connections are not so spaced, Q_{min} applies.

DIMENSIONS: Equipment and forming practices vary with different manufacturers, resulting in minor variations in some of these dimensions. These minor variations do not affect the published properties. Consult the manufacturer for actual weight per foot and actual dimensions.

TABLE 2



2 CHANNELS
WITH UNSTIFFENED FLANGES
BACK-TO-BACK



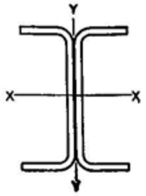
These Properties do not apply unless compression flanges are adequately braced laterally.

Nominal		Thickness	BEAM STRENGTH						DEFLECTION	
			$f_b = 18{,}000$ psi			$f_b = 27{,}000$ psi			Any Grade of Steel	
Size	Ga	t	S'_x	S'_y	K	S'_x	S'_y	K	I_x	I_y
In.	No.	In.	$In.^3$	$In.^3$		$In.^3$	$In.^3$		$In.^4$	$In.^4$
D x B										
8 x 4	10	.135	6.46	0.702	0.998	6.44	0.701	0.996	25.8	1.38
	12	.105	4.52	0.487	0.878	4.34	0.468	0.844	20.6	1.10
	14	.075	2.36	0.253	0.616	1.95	0.209	0.509	15.3	0.834
	16	.060	1.39	0.140	0.463	0.942	0.095	0.313	12.0	0.586
7 x 3	10	.135	4.30	0.363	1.000	4.30	0.363	1.000	15.1	0.510
	12	.105	3.54	0.313	1.000	3.54	0.313	1.000	12.4	0.464
	14	.075	2.18	0.182	0.844	2.08	0.172	0.799	9.08	0.315
	16	.060	1.51	0.114	0.748	1.37	0.103	0.677	7.08	0.211
6 x 3	10	.135	3.42	0.362	1.000	3.42	0.362	1.000	10.2	0.509
	12	.105	2.82	0.312	1.000	2.82	0.312	1.000	8.44	0.464
	14	.075	1.74	0.182	0.844	1.65	0.172	0.799	6.20	0.315
	16	.060	1.21	0.114	0.748	1.09	0.103	0.677	4.84	0.211
	18	.048	0.770	0.071	0.596	0.624	0.058	0.484	3.88	0.162
5 x 2½	12	.105	1.91	0.217	1.000	1.91	0.217	1.000	4.76	0.268
	14	.075	1.33	0.130	0.970	1.31	0.128	0.961	3.42	0.154
	16	.060	0.956	0.090	0.874	0.918	0.086	0.839	2.74	0.116
	18	.048	0.672	0.056	0.794	0.624	0.052	0.736	2.12	0.074
4 x 2¼	12	.105	1.33	0.195	1.000	1.33	0.195	1.000	2.66	0.229
	14	.075	0.968	0.140	0.945	0.952	0.138	0.929	2.04	0.180
	16	.060	0.692	0.090	0.874	0.664	0.086	0.839	1.58	0.116
	18	.048	0.480	0.060	0.755	0.436	0.054	0.686	1.27	0.088
3 x 2¼	12	.105	0.878	0.195	1.000	0.878	0.195	1.000	1.32	0.228
	14	.075	0.650	0.140	0.945	0.638	0.137	0.929	1.03	0.180
	16	.060	0.464	0.089	0.874	0.446	0.086	0.839	0.796	0.115
	18	.048	0.322	0.060	0.755	0.292	0.054	0.686	0.638	0.088
2 x 2¼	12	.105	0.500	0.194	1.000	0.500	0.194	1.000	0.500	0.227
	14	.075	0.378	0.140	0.945	0.372	0.137	0.929	0.400	0.179
	16	.060	0.270	0.089	0.874	0.260	0.086	0.839	0.308	0.115
	18	.048	0.188	0.060	0.755	0.170	0.054	0.686	0.248	0.088

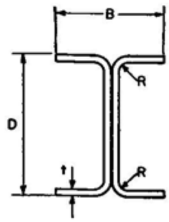
K is the ratio of the reduced allowable working stress f_c in the compression flange to the basic design stress, f_b (Sec. 3.2 p. 32); the factor K has been applied to the section moduli of the full section (Table No. 8, p. 72) to get the reduced section moduli, tabulated above. (See Reduced Working Stress, p. 2, also Reduced Properties, p. 3.)

By means of those reduced section moduli S'_x and S'_y, the flexural capacity or resisting moment of the section may be computed on the basis of the full basic design stress f_b. Thus, where the channels are adequately joined and the compression flanges are adequately braced laterally, the maximum allowable bending moment about the X-X axis equals f_b times S'_x, and about the Y-Y axis it equals f_b times S'_y.

TABLE 2



2 CHANNELS
WITH
UNSTIFFENED FLANGES
BACK-TO-BACK

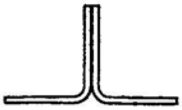


COLUMN PROPERTIES							DIMENSIONS					
Full Theoretical Outline			f_b = 18,000 psi		f_b = 27,000 psi		Depth	Flange	Thickness	Radius	Wt. per Foot	Nominal Size
Area	r_x	r_y	Q		Q		D	B	t	R		
Sq. In.	In.	In.	min.	max.	min.	max.	In.	In.	In.	In.	Lb.	In. D x B
3.10	2.89	0.669	0.814	0.985	0.761	0.979	8.0	3.934	.135	3/16	10.8	8 x 4
2.42	2.91	0.674	0.673	0.840	0.605	0.797	8.0	3.972	.105	3/16	8.46	
1.77	2.94	0.687	0.438	0.577	0.348	0.472	8.0	4.052	.075	3/32	6.16	
1.40	2.93	0.647	0.317	0.430	0.214	0.291	8.0	3.882	.060	3/32	4.88	
2.52	2.45	0.450	0.840	1.000	0.789	1.000	7.0	2.810	.135	3/16	8.78	7 x 3
2.00	2.48	0.481	0.776	0.970	0.720	0.960	7.0	2.972	.105	3/16	7.00	
1.45	2.50	0.466	0.581	0.787	0.513	0.732	7.0	2.928	.075	3/32	5.06	
1.14	2.49	0.429	0.477	0.684	0.405	0.606	7.0	2.758	.060	3/32	3.98	
2.24	2.13	0.476	0.881	1.000	0.843	1.000	6.0	2.810	.135	3/16	7.82	6 x 3
1.80	2.17	0.508	0.831	0.995	0.781	0.994	6.0	2.972	.105	3/16	6.26	
1.30	2.19	0.492	0.632	0.802	0.561	0.750	6.0	2.928	.075	3/32	4.52	
1.02	2.17	0.454	0.522	0.699	0.445	0.623	6.0	2.758	.060	3/32	3.56	
08.18	2.18	0.445	0.397	0.551	0.310	0.441	6.0	2.722	.048	3/32	2.86	
1.48	1.79	0.425	0.873	1.000	0.833	1.000	5.0	2.472	.105	3/16	5.16	5 x 2½
1.06	1.80	0.382	0.756	0.939	0.697	0.883	5.0	2.302	.075	3/32	3.68	
0.844	1.80	0.370	0.637	0.826	0.570	0.747	5.0	2.258	.060	3/32	2.94	
0.662	1.79	0.335	0.534	0.736	0.464	0.631	5.0	2.098	.048	3/32	2.30	
1.25	1.46	0.429	0.933	1.000	0.912	1.000	4.0	2.346	.105	3/16	4.34	4 x 2¼
0.924	1.49	0.441	0.812	0.945	0.758	0.929	4.0	2.428	.075	3/32	3.22	
0.724	1.48	0.400	0.709	0.851	0.642	0.795	4.0	2.258	.060	3/32	2.52	
0.578	1.48	0.390	0.582	0.720	0.502	0.626	4.0	2.222	.048	3/32	2.02	
1.04	1.13	0.470	1.000	1.000	1.000	1.000	3.0	2.346	.105	3/16	3.60	3 x 2¼
0.774	1.15	0.481	0.876	0.945	0.840	0.929	3.0	2.428	.075	3/32	2.70	
0.604	1.15	0.437	0.780	0.874	0.723	0.839	3.0	2.258	.060	3/32	2.10	
0.482	1.15	0.427	0.653	0.745	0.572	0.669	3.0	2.222	.048	3/32	1.68	
0.826	0.779	0.525	1.000	1.000	1.000	1.000	2.0	2.346	.105	3/16	2.88	2 x 2¼
0.624	0.800	0.536	0.945	0.945	0.929	0.929	2.0	2.428	.075	3/32	2.18	
0.484	0.799	0.488	0.855	0.874	0.815	0.839	2.0	2.258	.060	3/32	1.69	
0.386	0.802	0.477	0.714	0.755	0.641	0.686	2.0	2.222	.048	3/32	1.35	

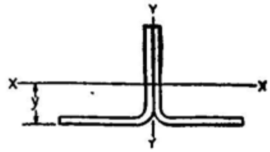
Q is the column factor defined in Sec. 3.6.1, Design Specification, p. 34. When the web-to-web connections are sufficiently closely spaced to cause the two webs to act as one, Q_{max} applies. When such connections are not so spaced, Q_{min} applies.

DIMENSIONS: Equipment and forming practices vary with different manufacturers, resulting in minor variations in some of these dimensions. These minor variations do not affect the published properties. Consult the manufacturer for actual weight per foot and actual dimensions.

TABLE 3



2 EQUAL LEG ANGLES
BACK-TO-BACK
UNSTIFFENED LEGS

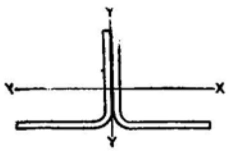


Nominal (One Angle) Size In.	Gage No.	Thickness In.	Beam Strength: Section Modulus Based on Full Theoretical Outline S_z In.^{3}	f_b = 18,000 psi: f_c psi	f_b = 18,000 psi: Mmax Comp./Tension In.-Lbs.	f_b = 18,000 psi: Mmax Comp./Tension In.-Lbs.	f_b = 27,000 psi: f_c psi	f_b = 27,000 psi: Mmax Comp./Tension In.-Lbs.	f_b = 27,000 psi: Mmax Comp./Tension In.-Lbs.	Deflection, Any Grade of Steel: y In.	Deflection, Any Grade of Steel: I_x In.^{4}
4 x 4	10	.135	1.164	9660	11260	20960	11020	12820	31440	1.069	3.430
3 x 3	10	.135	0.648	13720	8900	11680	18770	12180	17500	0.819	1.424
	12	.105	0.524	10230	5360	9420	12110	6340	14140	0.817	1.172
2½ x 2½	10	.135	0.446	15740	7040	8040	22650	10120	12060	0.694	0.814
	12	.105	0.364	12840	4660	6540	17090	6220	9800	0.692	0.676
2 x 2	10	.135	0.282	17770	5020	5080	26530	7500	7620	0.569	0.408
	12	.105	0.232	15440	3580	4180	22080	5120	6280	0.567	0.346
	14	.075	0.183	10260	1880	3300	12160	2220	4940	0.570	0.288
	16	.060	0.137	7880	1080	2480	7880	1080	3720	0.546	0.208

The properties of Table 3 may be used for flexural computations only when the angles are adequately joined and adequately braced laterally. Q is the column factor (Sec. 3.6.1, Design Specification, p. 34).

Where the vertical legs of the angles are in compression, M_{max} is based on the values of f_c (Sec. 3.2 of Design Specification) indicated; where the vertical legs of the angles are in tension M_{max} is based on f_b (tension) since the compression stress is always less than f_c for the sections listed.

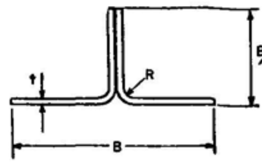
TABLE 3



2 EQUAL LEG ANGLES

BACK-TO-BACK

UNSTIFFENED LEGS

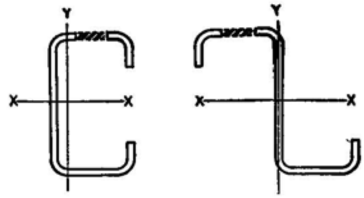


COLUMN PROPERTIES					DIMENSIONS				
Full Theoretical Outline			f_b = 18,000 psi	f_b = 27,000 psi	B	Thickness	Radius	Weight per Ft.	Nominal Size
Area	r_x	r_y	Q	Q		t	R		
Sq. In.	In.	In.			In.	In.	In.	Lbs.	In.
2.10	1.28	1.67	0.537	0.408	8.030	.135	3/16	7.32	4 x 4
1.56	0.955	1.26	0.762	0.695	6.030	.135	3/16	5.44	3 x 3
1.24	0.972	1.27	0.568	0.448	6.110	.105	3/16	4.32	
1.29	0.793	1.05	0.875	0.839	5.030	.135	3/16	4.50	2½ x 2½
1.03	0.811	1.07	0.713	0.633	5.110	.105	3/16	3.58	
1.02	0.632	0.850	0.987	0.983	4.030	.135	3/16	3.56	2 x 2
0.820	0.649	0.862	0.858	0.818	4.110	.105	3/16	2.86	
0.622	0.680	0.887	0.570	0.450	4.276	.075	3/32	2.16	
0.482	0.658	0.855	0.437	0.292	4.128	.060	3/32	1.68	

DIMENSIONS: Equipment and forming practices vary with different manufacturers, resulting in minor variations in some of these dimensions. These minor variations do not affect the published properties. Consult the manufacturer for actual weight per foot and actual dimensions.

TABLE 4

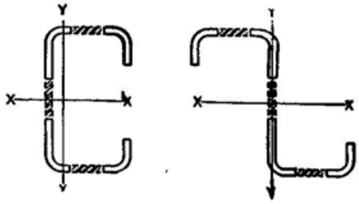
CHANNEL OR ZEE WITH STIFFENED FLANGES



These Properties do not apply unless sections are adequately braced laterally.

Nominal			BEAM STRENGTH						DEFLECTION		
			f_b = 18 000 psi			f_b = 27,000 psi			Any Grade of Steel		
			C or Z	C	Z	C or Z	C	Z	C or Z	C	Z
Size	Ga.	t	S_x	S_y	S_y	S_x	S_y	S_y	I_x	I_y	I_y
In.	No.	In.	$In.^3$	$In.^3$	$In.^3$	$In.^3$	$In.^3$	$In.^3$	$In.^4$	$In.^4$	$In.^4$
D x B											
12 x 3½	10	.135	9.37	This Property, which should rarely be required, depends on whether the web is in tension or in compression.	1.82	9.37	This Property, which should rarely be required, depends on whether the web is in tension or in compression.	1.82	56.2	This Property, which should rarely be required, depends on whether the web is in tension or in compression.	6.25
	12	.105	7.16		1.37	7.12		1.37	43.8 –.00000243 M		4.72
	14	.075	4.78		0.914	4.64		0.914	31.4 –.00001174 M		3.16
10 x 3½	10	.135	7.30		1.82	7.30		1.82	36.5		6.25
	12	.105	5.58		1.37	5.54		1.37	28.5 –.00000216 M		4.72
	14	.075	3.71		0.914	3.59		0.914	20.5 –.00001050 M		3.16
	16	.060	2.88		0.738	2.75		0.738	16.5 –.00001502 M		2.56
9 x 3¼	10	.135	6.04		1.62	6.04		1.62	27.2		5.14
	12	.105	4.64		1.15	4.64		1.15	21.0 –.00000029 M		3.67
	14	.075	3.12		0.808	3.03		0.808	15.3 –.00000812 M		2.59
	16	.060	2.39		0.614	2.29		0.614	12.2 –.00001204 M		1.98
8 x 3	10	.135	4.83		1.34	4.83		1.34	19.3		3.94
	12	.105	3.78		1.01	3.78		1.01	15.1		2.96
	14	.075	2.57		0.708	2.51		0.708	11.0 –.00000588 M		2.10
	16	.060	1.97		0.537	1.90		0.537	8.79 –.00000940 M		1.59
7 x 2¾	10	.135	3.75		1.10	3.75		1.10	13.1		2.94
	12	.105	2.98		0.873	2.98		0.873	10.4		2.35
	14	.075	2.08		0.614	2.04		0.614	7.66 –.00000378 M		1.67
	16	.060	1.58		0.464	1.53		0.464	6.10 –.00000704 M		1.26
6 x 2½	10	.135	2.81		0.874	2.81		0.874	8.42		2.13
	12	.105	2.24		0.697	2.24		0.697	6.72		1.71
	14	.075	1.63		0.527	1.61		0.527	5.03 –.00000188 M		1.30
	16	.060	1.24		0.397	1.21		0.397	4.01 –.00000484 M		0.981
5 x 2	10	.135	1.88		0.607	1.88		0.607	4.69		1.17
	12	.105	1.51		0.487	1.51		0.487	3.76		0.947
	14	.075	1.12		0.341	1.12		0.341	2.80		0.670
	16	.060	0.868		0.254	0.861		0.254	2.23 –.00000131 M		0.499
	18	.048	0.672		0.205	0.656		0.205	1.80 –.00000369 M		0.406
4 x 2	10	.135	1.38		0.606	1.38		0.606	2.76		1.17
	12	.105	1.11		0.486	1.11		0.486	2.22		0.947
	14	.075	0.832		0.341	0.832		0.341	1.66		0.670
	16	.060	0.646		0.254	0.641		0.254	1.33 –.00000112 M		0.499
	18	.048	0.499		0.205	0.487		0.205	1.08 –.00000316 M		0.406
3½ x 2	10	.135	1.15		0.606	1.15		0.606	2.01		1.17
	12	.105	0.927		0.486	0.927		0.486	1.62		0.947
	14	.075	0.699		0.341	0.699		0.341	1.22		0.670
	16	.060	0.543		0.254	0.538		0.254	0.979 –.00000102 M		0.499
	18	.048	0.419		0.205	0.408		0.205	0.795 –.00000288 M		0.406
3 x 1¾	12	.105	0.679		0.394	0.679		0.394	1.02		0.669
	14	.075	0.509		0.251	0.509		0.251	0.764		0.429
	16	.060	0.416		0.204	0.416		0.204	0.624		0.351
	18	.048	0.316		0.149	0.311		0.149	0.498 –.00000143 M		0.257

Note that X-X axis and Y-Y axis are not Principal Axes for the Zee. Properties of Table 4 may be used in flexural computations only when sections are adequately braced laterally and at supports. The section moduli about the Y-Y axis for the channels have not been tabulated. When the web is in compression the section modulus can be calculated in accordance with Sec. 2.3 of the Design



CHANNEL OR ZEE WITH STIFFENED FLANGES

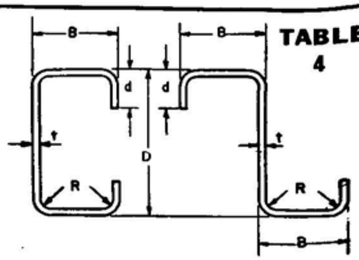


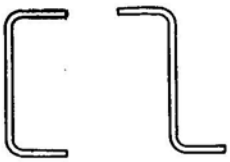
TABLE 4

COLUMN PROPERTIES							DIMENSIONS						
Full Theoretical Outline					f_b = 18000 psi	f_b = 27000 psi	C dimensions = Z dimensions						
C or Z	C or Z	C	Z	Z	C or Z	C or Z	Depth	Flange	Lip	Thick-ness	Radius	Wt. per Foot	Nominal Size
Area	r_x	r_y	r_y	r	Q	Q	D	B	d	t	R		
Sq. In.	In.	In.	In.	In.			In.	In.	In.	In.	In.	Lb.	In. D x B
2.70	4.56	1.22	1.52	1.06	0.737	0.693	12.0	3.50	1.0	.135	3/16	9.42	
2.10	4.57	1.21	1.50	1.04	0.665	0.621	12.0	3.50	0.9	.105	3/16	7.31	12 x 3½
1.49	4.59	1.19	1.46	1.01	0.539	0.487	12.0	3.50	0.7	.075	3/32	5.19	
2.43	3.87	1.25	1.60	1.07	0.800	0.757	10.0	3.50	1.0	.135	3/16	8.48	
1.89	3.89	1.25	1.58	1.05	0.728	0.683	10.0	3.50	0.9	.105	3/16	6.57	10 x 3½
1.34	3.91	1.23	1.54	1.02	0.594	0.538	10.0	3.50	0.7	.075	3/32	4.67	
1.08	3.92	1.23	1.54	1.02	0.530	0.473	10.0	3.50	0.7	.060	3/32	3.75	
2.23	3.49	1.17	1.52	1.00	0.829	0.787	9.0	3.25	1.0	.135	3/16	7.77	
1.71	3.50	1.15	1.47	0.968	0.768	0.726	9.0	3.25	0.8	.105	3/16	5.95	9 x 3¼
1.23	3.53	1.15	1.45	0.958	0.629	0.574	9.0	3.25	0.7	.075	3/32	4.28	
0.975	3.53	1.14	1.42	0.940	0.559	0.501	9.0	3.25	0.6	.060	3/32	3.40	
2.00	3.11	1.08	1.40	0.920	0.857	0.817	8.0	3.00	0.9	.135	3/16	6.97	
1.55	3.12	1.07	1.38	0.903	0.803	0.762	8.0	3.00	0.8	.105	3/16	5.40	8 x 3
1.12	3.15	1.08	1.37	0.893	0.670	0.616	8.0	3.00	0.7	.075	3/32	3.89	
0.885	3.15	1.06	1.34	0.876	0.598	0.540	8.0	3.00	0.6	.060	3/32	3.08	
1.77	2.72	0.982	1.29	0.838	0.886	0.850	7.0	2.75	0.8	.135	3/16	6.17	
1.39	2.74	0.996	1.30	0.837	0.839	0.799	7.0	2.75	0.8	.105	3/16	4.86	7 x 2¾
1.00	2.76	0.999	1.29	0.828	0.718	0.667	7.0	2.75	0.7	.075	3/32	3.50	
0.795	2.77	0.986	1.26	0.810	0.641	0.584	7.0	2.75	0.6	.060	3/32	2.77	
1.54	2.34	0.885	1.17	0.755	0.913	0.886	6.0	2.50	0.7	.135	3/16	5.37	
1.21	2.35	0.900	1.19	0.754	0.875	0.838	6.0	2.50	0.7	.105	3/16	4.23	6 x 2½
0.891	2.38	0.921	1.21	0.760	0.775	0.729	6.0	2.50	0.7	.075	3/32	3.10	
0.705	2.39	0.910	1.18	0.744	0.694	0.639	6.0	2.50	0.6	.060	3/32	2.46	
1.27	1.92	0.715	0.960	0.625	0.948	0.932	5.0	2.00	0.7	.135	3/16	4.43	
1.00	1.94	0.729	0.971	0.624	0.907	0.877	5.0	2.00	0.7	.105	3/16	3.50	
0.726	1.96	0.733	0.961	0.615	0.836	0.796	5.0	2.00	0.6	.075	3/32	2.53	5 x 2
0.573	1.97	0.721	0.934	0.598	0.766	0.721	5.0	2.00	0.5	.060	3/32	2.00	
0.461	1.98	0.727	0.938	0.598	0.689	0.636	5.0	2.00	0.5	.048	3/32	1.61	
1.14	1.56	0.727	1.02	0.614	1.000	1.000	4.0	2.00	0.7	.135	3/16	3.96	
0.900	1.57	0.740	1.03	0.612	0.954	0.939	4.0	2.00	0.7	.105	3/16	3.14	
0.651	1.60	0.745	1.01	0.603	0.896	0.863	4.0	2.00	0.6	.075	3/32	2.27	4 x 2
0.513	1.61	0.735	0.987	0.587	0.832	0.789	4.0	2.00	0.5	.060	3/32	1.79	
0.413	1.62	0.740	0.991	0.586	0.754	0.700	4.0	2.00	0.5	.048	3/32	1.44	
1.07	1.37	0.730	1.05	0.598	1.000	1.000	3.5	2.00	0.7	.135	3/16	3.73	
0.847	1.38	0.744	1.06	0.596	0.982	0.976	3.5	2.00	0.7	.105	3/16	2.95	
0.613	1.41	0.750	1.05	0.587	0.921	0.896	3.5	2.00	0.6	.075	3/32	2.14	3½ x 2
0.483	1.42	0.740	1.02	0.572	0.866	0.826	3.5	2.00	0.5	.060	3/32	1.68	
0.389	1.43	0.745	1.02	0.571	0.789	0.736	3.5	2.00	0.5	.048	3/32	1.36	
0.742	1.17	0.655	0.949	0.529	1.000	1.000	3.0	1.75	0.7	.105	3/16	2.59	
0.523	1.21	0.647	0.906	0.508	0.944	0.926	3.0	1.75	0.5	.075	3/32	1.82	3 x 1¾
0.423	1.22	0.654	0.911	0.506	0.916	0.889	3.0	1.75	0.5	.060	3/32	1.47	
0.331	1.23	0.642	0.881	0.491	0.843	0.798	3.0	1.75	0.4	.048	3/32	1.16	

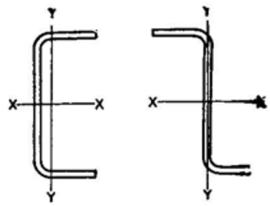
Specification, p. 27: When the web becomes a tension flange, the section modulus is that of the full section (Table 7).

Q is the column factor (Sec. 3.6.1). The areas and moments of inertia to be used for lateral bracing determination, when the sections are used as compression members, are those of the full section (Table 7).

TABLE 5



CHANNEL OR ZEE
WITH
UNSTIFFENED FLANGES



These Properties do not apply unless sections are adequately braced laterally.

			BEAM STRENGTH								DEFLECTION		
			f_b = 18,000 psi				f_b = 27,000 psi				Any Grade of Steel		
Nominal		Th'kness	C or Z	C	Z	C or Z	C or Z	C	Z	C or Z	C or Z	C	Z
Size	Gage	t	S'_x	S_y	S'_y	K	S'_x	S_y	S_y	K	I_x	I_y	I_y
In.	No.	In.	In.^{3}	In.^{3}	In.^{3}		n.^{3}	In.^{3}	In.^{3}		In.^{4}	In.^{4}	In.^{4}
D x B				This property, rarely required, varies depending on whether the web is in tension or in compression.				This property, rarely required, varies depending on whether the web is in tension or in compression.				This property, rarely required, varies depending on whether the web is in tension or in compression.	
8 x 2	10	.135	3.23		.360	0.998	3.22		.359	0.996	12.9		.685
	12	.105	2.26		.249	0.878	2.17		.239	0.844	10.3		.548
	14	.075	1.18		.129	0.616	0.974		.106	0.509	7.66		.416
	16	.060	0.696		.071	0.463	0.471		.048	0.313	6.02		.293
7 x 1½	10	.135	2.15		.186	1.000	2.15		.186	1.000	7.54		.249
	12	.105	1.77		.160	1.000	1.77		.160	1.000	6.18		.229
	14	.075	1.09		.093	0.844	1.04		.088	0.799	4.54		.157
	16	.060	0.757		.058	0.748	0.685		.053	0.677	3.54		.105
6 x 1½	10	.135	1.71		.186	1.000	1.71		.186	1.000	5.12		.249
	12	.105	1.41		.160	1.000	1.41		.160	1.000	4.22		.229
	14	.075	0.872		.093	0.844	0.827		.088	0.799	3.10		.157
	16	.060	0.603		.058	0.748	0.546		.053	0.677	2.42		.105
	18	.048	0.885		.036	0.596	0.312		.029	0.484	1.94		.081
5 x 1¼	12	.105	0.953		.111	1.000	0.953		.111	1.000	2.38		.132
	14	.075	0.663		.066	0.970	0.656		.066	0.961	1.71		.076
	16	.060	0.478		.046	0.874	0.459		.044	0.839	1.37		.058
	18	.048	0.336		.029	0.794	0.312		.027	0.736	1.06		.037
4 x 1⅛	12	.105	0.663		.101	1.000	0.663		.101	1.000	1.33		.113
	14	.075	0.484		.072	0.945	0.476		.070	0.929	1.02		.089
	16	.060	0.346		.046	0.874	0.332		.044	0.839	0.792		.057
	18	.048	0.240		.030	0.755	0.218		.028	0.686	0.635		.044
3 x 1⅛	12	.105	0.439		.101	1.000	0.439		.101	1.000	0.658		.113
	14	.075	0.325		.072	0.945	0.319		.070	0.929	0.515		.089
	16	.060	0.232		.046	0.874	0.223		.044	0.839	0.398		.057
	18	.048	0.161		.030	0.755	0.146		.028	0.686	0.319		.044
2 x 1⅛	12	.105	0.250		.100	1.000	0.250		.100	1.000	0.250		.113
	14	.075	0.189		.072	0.945	0.186		.070	0.929	0.200		.089
	16	.060	0.135		.046	0.874	0.130		.044	0.839	0.154		.057
	18	.048	0.094		.030	0.755	0.085		.028	0.686	0.124		.044

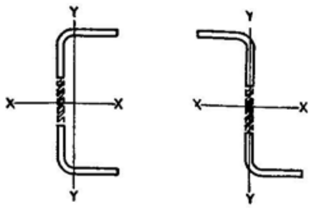
Note that X-X axis and Y-Y axis are not Principal Axes for the Zee.

The properties of Table 5 may be used in flexural computations only when the sections are adequately braced laterally throughout their entire length as well as at supports.

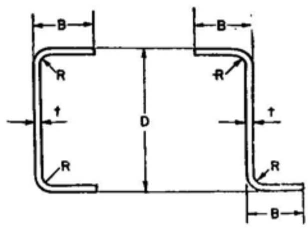
K is the ratio of the allowable working stress f_c in the compression flange to the basic design stress, f_b; it also is the factor applied to the section moduli of the full section (Table 8 of p. 72) to get the reduced section moduli, tabulated above. See Reduced Working Stresses, p. 2, also Reduced Properties, p. 3.

By means of these reduced section moduli S'_x and S'_y the flexural capacity or resisting moment of the section may be computed on the basis of the full basic design stress, f_b. Thus, where the channels are adequately braced laterally, the maximum allowable bending moment about the X-X axis equals f_b times S'_x, and about the Y-Y axis it equals f_b times S'_y.

TABLE 5



CHANNEL OR ZEE WITH UNSTIFFENED FLANGES



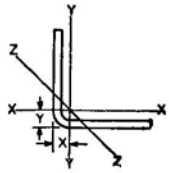
COLUMN PROPERTIES							DIMENSIONS					Nominal Size
Full Theoretical Outline					$f_b =$ 18,000 psi	$f_b =$ 27,000 psi	[dimensions = Z dimensions					
[OR Z	[OR Z	[	Z	Z	[OR Z	[OR Z	Depth	Flange	Thickness	Radius	Wt. per Foot	
Area	r_x	r_y	r_y	r min.	Q	Q	D	B	t	R		
Sq. In.	In.	In.	In.	In.			In.	In.	In.	In.	Lb.	In.
												D x B
1.55	2.89	0.548	0.666	0.503	0.814	0.761	8.0	1.967	.135	3/16	5.38	
1.21	2.91	0.559	0.672	0.503	0.673	0.605	8.0	1.986	.105	3/16	4.23	8 x 2
0.884	2.94	0.577	0.686	0.510	0.438	0.348	8.0	2.026	.075	3/32	3.08	
0.699	2.93	0.549	0.647	0.483	0.317	0.214	8.0	1.941	.060	3/32	2.44	
1.26	2.45	0.368	0.445	0.352	0.840	0.789	7.0	1.405	.135	3/16	4.39	
1.00	2.48	0.399	0.478	0.370	0.776	0.720	7.0	1.486	.105	3/16	3.50	7 x 1½
0.725	2.50	0.395	0.465	0.360	0.581	0.513	7.0	1.464	.075	3/32	2.53	
0.572	2.49	0.368	0.428	0.333	0.477	0.405	7.0	1.379	.060	3/32	1.99	
1.12	2.13	0.382	0.471	0.363	0.881	0.843	6.0	1.405	.135	3/16	3.91	
0.898	2.17	0.415	0.505	0.380	0.831	0.781	6.0	1.486	.105	3/16	3.13	
0.650	2.19	0.410	0.491	0.370	0.632	0.561	6.0	1.464	.075	3/32	2.26	6 x 1½
0.512	2.17	0.383	0.453	0.343	0.522	0.445	6.0	1.379	.060	3/32	1.78	
0.409	2.18	0.378	0.444	0.336	0.397	0.310	6.0	1.361	.048	3/32	1.43	
0.741	1.79	0.343	0.422	0.319	0.873	0.833	5.0	1.236	.105	3/16	2.58	
0.528	1.80	0.316	0.380	0.291	0.756	0.697	5.0	1.151	.075	3/32	1.84	5 x 1¼
0.422	1.80	0.311	0.369	0.282	0.637	0.570	5.0	1.129	.060	3/32	1.47	
0.331	1.79	0.284	0.334	0.257	0.534	0.464	5.0	1.049	.048	3/32	1.15	
0.623	1.46	0.337	0.426	0.311	0.933	0.912	4.0	1.173	.105	3/16	2.17	
0.462	1.49	0.355	0.439	0.318	0.812	0.758	4.0	1.214	.075	3/32	1.61	4 x 1⅛
0.362	1.48	0.327	0.399	0.291	0.709	0.642	4.0	1.129	.060	3/32	1.26	
0.289	1.48	0.322	0.390	0.284	0.582	0.502	4.0	1.111	.048	3/32	1.01	
0.518	1.13	0.354	0.467	0.317	1.000	1.000	3.0	1.173	.105	3/16	1.80	
0.387	1.15	0.372	0.480	0.324	0.876	0.840	3.0	1.214	.075	3/32	1.35	3 x 1⅛
0.302	1.15	0.344	0.436	0.298	0.780	0.723	3.0	1.129	.060	3/32	1.05	
0.241	1.15	0.339	0.426	0.291	0.653	0.572	3.0	1.111	.048	3/32	0.841	
0.413	0.779	0.369	0.522	0.305	1.000	1.000	2.0	1.173	.105	3/16	1.44	
0.312	0.800	0.387	0.534	0.312	0.945	0.929	2.0	1.214	.075	3/32	1.09	2 x 1⅛
0.242	0.799	0.360	0.487	0.291	0.855	0.815	2.0	1.129	.060	3/32	0.843	
0.193	0.802	0.356	0.476	0.284	0.714	0.641	2.0	1.111	.048	3/32	0.673	

Q is the column factor defined in Sec. 3.6.1, Design Specification, p. 34.

DIMENSIONS: Equipment and forming practices vary with different manufacturers, resulting in minor variations in some of these dimensions. These minor variations do not affect the published properties. Consult the manufacturer for actual weight per foot and actual dimensions.

TABLE 6

EQUAL LEG ANGLE
WITH UNSTIFFENED LEGS



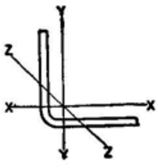
Nominal			Beam Strength								Deflection
			Section Modulus (Full unreduced section) $S_x = S_y$	f_b = 18,000 psi			f = 27,000 psi				Any Grade of Steel $I_x = I_y$
				M_{max}		M_{max}	M_{max}		M_{max}		
Size	Gage	Thickness		f_c	Comp. / Tension	Comp. / Tension	f_c	Comp. / Tension	Comp. / Tension	x = y	
In.	No.	In.	$In.^3$	psi	In.-Lbs.	In.-Lbs.	psi	In.-Lbs.	In.-Lbs.	In.	$In.^4$
4 x 4	10	.135	0.582	9660	5630	10480	11020	6410	15720	1.069	1.715
3 x 3	10	.135	0.324	13720	4450	5840	18770	6090	8750	0.819	0.712
	12	.105	0.262	10230	2680	4710	12110	3170	7070	0.817	0.586
2½ x 2½	10	.135	0.223	15740	3520	4020	22650	5060	6030	0.694	0.407
	12	.105	0.182	12840	2330	3270	17090	3110	4900	0.692	0.338
2 x 2	10	.135	0.141	17770	2510	2540	26530	3750	3810	0.569	0.204
	12	.105	0.116	15440	1790	2090	22080	2560	3140	0.567	0.173
	14	.075	0.0916	10260	940	1650	12160	1110	2470	0.570	0.144
	16	.060	0.0687	7880	540	1240	7880	540	1860	0.546	0.104

The properties of Table 6 apply only when the sections are adequately braced laterally.

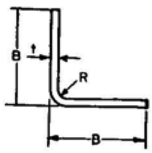
When single angles are used as compression members due regard must be given to any eccentricity of loading, and allowable unit stress shall be calculated in accordance with Sec. 3.7, of the Design Specification, p. 35. Neglect of this feature can prove dangerous.

Where the vertical legs of the angles are in compression, M_{max} is based on the values of f_c (See 3.2 of Design Specification) indicated; where the vertical legs of the angles are in tension M_{max} is based on f_b (tension) since the compression stress is always less than f_c for the sections listed.

TABLE 6



EQUAL LEG ANGLE
WITH UNSTIFFENED LEGS



COLUMN PROPERTIES				DIMENSIONS				
Full Theoretical Outline		f_b = 18,000 psi	f_b = 27,000 psi	Leg	Thickness	Radius	Weight per Foot	Nominal Size
Area	r_z (min.)	Q	Q	B	t	R		
Sq. In.	In.			In.	In.	In.	Lbs.	In.
1.05	0.794	0.537	0.408	4.015	.135	3/16	3.66	4 x 4
0.781	0.589	0.762	0.695	3.015	.135	3/16	2.72	3 x 3
0.620	0.601	0.568	0.448	3.055	.105	3/16	2.16	
0.646	0.487	0.875	0.839	2.515	.135	3/16	2.25	2½ x 2½
0.515	0.499	0.713	0.633	2.555	.105	3/16	1.79	
0.511	0.385	0.987	0.983	2.015	.135	3/16	1.78	2 x 2
0.410	0.396	0.858	0.818	2.055	.105	3/16	1.43	
0.311	0.423	0.570	0.450	2.138	.075	3/32	1.08	
0.241	0.409	0.437	0.292	2.064	.060	3/32	0.840	

Q is the column factor defined in Sec. 3.6.1, of the Design Specification, p. 34.

DIMENSIONS: Equipment and forming practices vary with different manufacturers, resulting in minor variations in some of these dimensions. These minor variations do not affect the published properties. Consult the manufacturer for actual weight per foot and actual detailed dimensions.

PART III

EXCERPTS FROM SPECIFICATION FOR THE DESIGN OF LIGHT GAGE STEEL STRUCTURAL MEMBERS

(Published by American Iron and Steel Institute, April, 1946)

SECTION 1. GENERAL

1.1—SCOPE

This Specification shall apply to the design of structural members cold formed to shape from sheet or strip steel less than 3/16 inch thick and used for load-carrying purposes in buildings.

Nothing herein is intended to conflict with provisions of the Specifications issued by the American Institute of Steel Construction for the Design, Fabrication, and Erection of Structural Steel for Buildings nor with the Standard Specifications for Steel Joist Construction as adopted by the Steel Joist Institute.

1.2—MATERIAL

Steel shall conform to the Tentative Specifications of the American Society for Testing Materials for Light Gage Structural Quality Flat Rolled Carbon Steel, Serial Designations A245-T and A-246-T, as amended to date, except as otherwise provided herein.* The terms C, B, and A when used herein to designate grades of steel shall refer to grades provided by those Tentative ASTM Specifications.

Steel of higher strength than is covered by the above-mentioned ASTM specifications may be used at the unit stresses hereinafter specified for "other" grades of steel provided the design is based upon the minimum properties of those grades of steel as guaranteed by the manufacturer. It is the intent of this Specification to permit the use of high-strength steels of suitable properties for purposes coming within the scope of this Specification, but not to permit the use of ordinary carbon steels at unit stresses higher than those specified in Section 3 for Grade C material.

SECTION 2. DESIGN PROCEDURE

2.1—PROCEDURE

All computations for safe load, stress, deflection and the like shall be in accordance with conventional methods of structural design except as otherwise specified herein.

2.2—DEFINITIONS

Where the following terms appear in this Specification they shall have the meaning herein indicated:

(a) *Stiffened Compression Elements.* The term "stiffened compression elements" shall mean flat compression elements (i.e., plane compression flanges of flexural members and plane webs and flanges of compression members) of

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* It is appropriate also to include the new ASTM Specification A303-47T for Hot-Rolled Strip of Structural Quality.

which *both* edges parallel to the direction of stress are stiffened by connection to a stiffening means (i.e., web, flange, stiffening lip, intermediate stiffener, or the like) conforming to the requirements of Sec. 2.3.2.

(b) *Unstiffened Compression Elements.* Any flat element which is stiffened at only *one* edge parallel to the direction of stress shall be considered an "unstiffened" element.

(c) *Flat-Width Ratio.* The flat-width ratio is the ratio, w/t, of the flat width, w, exclusive of edge fillets, of a single flat element to the thickness, t, of such element. In the case of sections such as I-, T-, channel- and Z-shaped sections, the width, w, is the width of the flat projection of flange from web, exclusive of fillets, and of any stiffening lip that may be at the outer edge of the flange. In the case of *multiple-web* sections such as inverted U-type or box-shaped sections, the width, w, is the flat width of flange between adjacent webs, exclusive of fillets.

(d) *Effective Design Width.* Where the flat width, w, of an element is reduced for design purposes, the reduced design width, b, is termed the "effective width," or the "effective design width." This "effective design width" is determined in accordance with Sec. 2.3.1 and 2.3.5. See sketches of "Effective Cross Sections," p. 5.

(e) *Box Type Sections.* See Figure 3 of "Effective Cross Sections," p. 5.

(f) *U-Type Section* (inverted). See Figure 4 of "Effective Cross Sections," p. 5.

(g) *Multiple-Stiffened Elements.* A multiple-stiffened element is one that is stiffened by means of one or more intermediate ribs or stiffeners which are parallel to the direction of stress, and which conform to the requirements of Sec. 2.3.2, dividing the element into a number of narrower sub-elements each of which shall be considered individually. See Figure 5 of "Effective Cross Sections," p. 5.

2.3—PROPERTIES OF SECTIONS

Properties of sections (cross-sectional area, moment of inertia, section modulus, radius of gyration, etc.) shall be determined in accordance with conventional methods of structural design and shall be based on the full cross-section of the members (or net section where the use of a net section is customary) except where the use of a reduced cross-section, or "effective design width," is required by the provisions of Sec. 2.3.1 and 2.3.5 of this Specification.

2.3.1 Properties of Stiffened Compression Elements.

Effective Design Width—In computing the properties of sections of flexural members and in computing the values of "Q" (Sec. 3.6.1) for compression members, the flat width, w, of any stiffened compression element having a flat-width ratio larger than 25 shall be considered as being reduced for design purposes to an *effective design width,* b, determined in accordance with the following formulas. (The application of these formulas is facilitated by the use of Charts 3A to 3D, pp. 7-10, or by the use of Table 2.3.1, p. 29, when the actual stress is 18,000 lb. per sq. in. Charts 3A to 3D may be used for any grade of steel and for any stress; Table 2.3.1 is applicable only for Grade C steel and for $f = 18{,}000$ lb. per sq. in.).

The curves in Charts 3A to 3D, and the values in Table 2.3.1, are based on the following formulas: **

For load determinations:

For w/t equal to or less than M,* where

$$M = \frac{(17.5 \times 10^6) + \sqrt{(17.5 \times 10^6)^2 - (436 \times 10^6)(7600\sqrt{f} - 25f)}}{7600\sqrt{f} - 25f}$$

$$\frac{b}{t} = \frac{(17.5 \times 10^6)(w/t - 25)}{fM^2} + 25$$

For w/t greater than M,*

$$\frac{b}{t} = \frac{7600}{\sqrt{f}}\left[1 - \frac{2300}{(w/t)\sqrt{f}}\right]$$

For deflection determinations:

For w/t equal to or less than N,* where

$$N = \frac{(32.2 \times 10^6) + \sqrt{(32.2 \times 10^6)^2 - (805 \times 10^6)(10{,}320\sqrt{f} - 25f)}}{10{,}320\sqrt{f} - 25f},$$

$$\frac{b}{t} = \frac{(32.2 \times 10^6)(w/t - 25)}{fN^2} + 25$$

For w/t greater than N,*

$$\frac{b}{t} = \frac{10{,}320}{\sqrt{f}}\left[1 - \frac{3120}{(w/t)\sqrt{f}}\right]$$

Where w/t = flat width ratio (see Sec. 2.2)
b = effective design width
f = unit stress in the compression element
(p.s.i.) computed on the basis of the *effective design width.*

That portion of the width considered removed to arrive at the *effective design width* shall be located symmetrically about the center line of the elements. See sketches of "Effective Cross Sections," p. 5.

** In using Charts 3A to 3D the effective design width should be determined from the actual unit stress in the compression element. In those cases where the compression flange is larger than the tension flange or where the unit compression stress is less than the maximum allowable design stress, the properties of the section can be determined only approximately by using the full design stress. For accurate determinations of properties and effective design width the actual stress must be used since the effective width varies with the unit stress and increases as the latter decreases.

It should be noted that where the w/t ratio is high (about 250 or more) there may be noticeable deformation of the compression element. However, this occurs without detriment to the load-carrying capacity as determined in accordance with this Specification.

* When the actual compression, f, is 18,000 p.s.i., the values for M and N may be obtained from Table 2.3.1.

TABLE 2.3.1

Ratio b/t of Effective Design Width to Thickness of Stiffened Compression Elements, for f = 18,000 lb. per sq. in.

w/t	b/t for Load	b/t for Deflection	w/t	b/t for Load	b/t for Deflection
25	25.0	25.0	120	48.5	62.8
30	27.5	28.4	140	49.7	64.2
35	30.0	31.5	160	50.5	65.8
40	32.4	34.8	180	51.2	67.0
43.8*	34.5	—	200	51.8	68.0
45	35.1	38.0	225	52.3	69.0
50	37.3	41.3	250	52.8	69.8
52.4**	—	42.8	275	53.1	70.4
60	40.5	47.2	300	53.4	71.0
70	42.8	51.4	350	53.9	71.9
80	44.5	55.3	400	54.2	72.5
90	45.9	57.1	450	54.5	72.9
100	47.0	59.1	500	54.7	73.4

* *M* value (f = 18,000 lb. per sq. in.) see Sec. 2.3.1
** *N* value (f = 18,000 lb. per sq. in.) see Sec. 2.3.1

2.3.2 Stiffeners for Compression Elements.

2.3.2.1 Minimum Properties and Dimensions of Edge Stiffeners.

In order that a compression element may be considered a "stiffened compression element" it shall be stiffened at each longitudinal edge, running parallel to the direction of stress, by a web, lip, or other stiffening means, having the following minimum moment of inertia:

$$I_{min} = 1.83t^4 \sqrt{(w/t)^2 - 144} \quad \text{(See Table 2.3.2.1)}$$

where w/t = flat-width ratio of stiffened element

I = Minimum moment of inertia of stiffener (of any shape) about its own centroidal axis parallel to the stiffened element.

For *intermediate stiffeners* which serve to stiffen compression elements on both sides of the stiffener, the moment of inertia shall be twice that specified for I_{min} above.

Where the stiffener consists of a simple lip bent at right angles to the stiffened element, the required depth d of such lip may be approximated by using the formula

$$d = 2.8t \sqrt[6]{(w/t)^2 - 144} \qquad \text{(See Table 2.3.2.1)}$$

TABLE 2.3.2.1

Minimum Properties of Stiffening Elements

w/t	I	d	w/t	I	d
12 or less	0	0	20	$29.3t^4$	7.1t
			25	$40.2t^4$	7.8t
13	$9.2t^4$	4.8t	30	$50.4t^4$	8.5t
14	$13.2t^4$	5.4t	40	$69.9t^4$	9.4t
16	$19.4t^4$	6.2t	50	$89.0t^4$	10.2t
18	$24.6t^4$	6.7t	60	$107.6t^4$	10.9t
			Over 60	$1.83t^3w$	$2.8\sqrt[3]{t^2w}$

2.3.2.2 Spacing of Connections.

The longitudinal spacing of rivets or welds connecting a non-integral stiffener to a compression element shall not exceed one-third of the flat width of the stiffened element.

2.3.3 Maximum Allowable Flat-Width Ratios.

Maximum allowable flat-width ratios, w/t, shall be as follows:

(a) Stiffened compression element with one longitudinal edge connected to a web or flange element 60

(b) Compression element stiffened at both longitudinal edges by connection to a web (U-type, box-type sections) or flange 500

(c) Unstiffened compression element 60

Note: The use of unstiffened compression elements having flat-width ratios larger than 30, but not exceeding 60, may result in noticeable distortion of the free edges of such elements without detriment to the ability of the member to support load. Under the limiting stresses prescribed in Sec. 3.2 such distortion can be expected generally to be in the order of less than 1/16 inch.

(d) *Multiple stiffened elements:* The appropriate maximum allowable flat-width ratios of sub-divisions (a), (b) and (c) of this Section shall apply to the individual sub-elements. The total width of the composite multiple elements shall be limited only by the provisions of sub-division (e) of this Section where applicable.

(e) *Unusually Wide Flanges:* Where a flange is unusually wide and it is desired to limit the maximum amount of curling or movement of the flange toward the neutral axis, the following formula applies to compression and tension flanges, either stiffened or unstiffened:

$$w_{max} = \sqrt{\frac{1{,}800{,}000th}{f_{av}}} \times \sqrt[4]{D}$$

where w_{max} = the width or projection in inches of flange beyond the web fillet for I-beams; or half of the distance between web fillets for box- or U-type beams

t	=	thickness of flange or compression element in inches
h	=	depth of beam in inches
D	=	the amount of curling in percent of the depth, h, (e.g. if 5% of the beam depth is the curling limitation, $D = 5$ not .05)
f_{av}	=	the *average* stress in the full, unreduced flange width. [Where members are designed by the *effective design width* procedure, the average stress = the maximum stress x (the ratio of the effective design width to the actual width)]

2.3.4 Laterally Unbraced Compression Flanges.

For closed box-type sections the ratio of the laterally unsupported flange length, L, to the gross width of section shall not exceed 75

2.3.5 Unusually Short Spans Supporting Concentrated Loads.

Where the span of a beam is less than 30 times the width of its flange projection from the web, and it carries a concentrated load, or loads spaced farther apart than twice the width of flange projection, the effective design width of the tension flange shall be limited as follows:

TABLE 2.3.5

Short, Wide Tension Flanges

Ratio of Effective Design Width to Actual Width

L/w	Ratio	L/w	Ratio
30	1.00	14	.80
25	0.95	12	.75
20	.90	10	.70
18	.86	8	.65

In Table 2.3.5 above:

L = full span for simple spans; or the distance between inflection points for continuous beams; or twice the length of cantilever beams, in inches.

w = width of flange projection beyond the web fillet for I-beam and similar sections or half the flat width of flanges of box- or U-type sections, in inches.

For tension flanges of I-beams stiffened by bent-over lips at the outer edges, w shall be taken as the sum of the flange projection beyond the web plus the depth of the bent-over lip.

SECTION 3. ALLOWABLE DESIGN STRESSES

The maximum allowable unit stresses to be used in design shall be as follows:

3.1—BASIC DESIGN STRESS

Tension on the net section of tension members, and tension and compression, f_b, on the extreme fibers of flexural members shall not exceed the values specified below except as otherwise specifically provided in this Section.

Grade of Steel	Min. Yield Point lb. per sq. in.	f_b (lbs. per sq. in.)
C	33,000	18,000
B	30,000	16,500
A	25,000	13,500
Other	f_b = Minimum specified yield point/1.85	

3.2—COMPRESSION ON UNSTIFFENED ELEMENTS

Compression, f_c, in pounds per square inch, on flat unstiffened elements:

(a) For w/t not greater than 12, $f_c = f_b$

(b) For w/t greater than 12 but not over 30:

$$f_c = [1.67\ f_b - 5430] - (1/18)\ (f_b - 8150)\ w/t$$

where w/t = flat-width ratio, (See Sec. 2.2)

(Values in accordance with the above formula are given in Table 3.2(b) below.)

TABLE 3.2(b)
Allowable Design Stresses
ASTM A245-T and A246-T Grades of Steel
For w/t Ratios From 12 to 30

w/t	Grade C	Grade B	Grade A
12	18,000	16,500	13,500
14	16,910	15,580	12,910
16	15,810	14,650	12,310
18	14,720	13,720	11,720
20	13,630	12,790	11,130
22	12,530	11,860	10,530
24	11,440	10,940	9,940
26	10,340	10,010	9,340
28	9,250	9,080	8,750
30	8,150	8,150	8,150

(c) For w/t over 30 but not over 60: $f_c = 12{,}600 - 148.5\ (w/t)$*

(Values in accordance with the above formula are given in Table 3.2(c) below. See Examples in Part V.)

TABLE 3.2(c)
Allowable Design Stresses**
For w/t Ratios From 30 to 60 (All Grades of Steel)

Ratio w/t	f	Ratio w/t	f	Ratio w/t	f
30	8150	40	6660	50	5180
32	7850	42	6360	52	4880
34	7550	44	6070	54	4580
36	7250	46	5770	56	4280
38	6960	48	5470	58	3990
				60	3690

* Angle sections are subject to torsional buckling unless braced in a manner which prevents twisting and should be investigated accordingly. For single or double angles not braced against twist the allowable unit stress for w/t between 30 and 60 shall not exceed $7{,}330{,}000 \div (w/t)^2$.

** Flanges or compression elements having ratios of w/t larger than 30 may show noticeable distortion of the free edges under maximum working stress without detriment to the ability of the member to support load. For the stresses and range of ratios provided in the formula above and in Table 3.2(c) the distortion generally will be of the order of less than 1/16 inch.

For ratios beyond w/t = 60 distortion of the flanges is likely to be so pronounced as to render the section structurally undesirable unless load and stress are limited to such a degree as to render such use uneconomical.

3.3—LATERALLY UNBRACED COMPRESSION FLANGES

Maximum compression, f'_c, in pounds per square inch, on extreme fibers of laterally unsupported compression flanges of straight I-shaped flexural members, (not including multiple-web deck, U- and closed box-type members and curved or arch members), shall not exceed the allowable stress as specified in Sec. 3.1 or 3.2 nor the following maximum stresses:

$$f'_c = \frac{250{,}000{,}000}{(L/r_y)^2} \qquad L = r_y \sqrt{\frac{250{,}000{,}000}{f'_c}}$$

where L is the unbraced length of the member, and r_y is the radius of gyration of the entire section of the member about its gravity axis parallel to the web; both in inch units. In the above formula for L, the maximum allowable unbraced length, the denominator of the fraction under the radical shall be f_c, the maximum allowable unit design stress in compression, or the actual unit compression stress, f'_c, where the latter value is less than f_c.

3.4—ALLOWABLE WEB SHEAR

The maximum average shear stress, v, in pounds per square inch, on the gross area of a flat web shall not exceed:

$$v = \frac{64{,}000{,}000}{(h/t)^2} \text{ with a maximum of } 2/3\ f_b.$$

In the above formula, $t =$ web thickness, $h =$ clear distance between flanges, and $f_b =$ basic working stress as specified in Sec. 3.1.

Where the web consists of two or more sheets, each sheet shall be considered as a separate member carrying its share of the stress. If, in such cases, the sheets are joined together by continuous welds or by rows of spot welds parallel to the flanges, "h" shall be the vertical distance between the rows of welds or between a row of welds and the flange, whichever is the greater, (rather than the distance between the flanges) provided the longitudinal spacing of welds along each row of welds does not exceed h/3.

Tabular values are given in Table 3.4, for Grade C Steel.

TABLE 3.4

Maximum Allowable Web Shear on Flat Webs

Grade C Steel

$$v = \frac{64{,}000{,}000}{(h/t)^2} \text{ with maximum } 2/3\ f_b = 12{,}000 \text{ lb. per sq. in.}$$

h/t	v	h/t	v	h/t	v
73	12,000	110	5,290	150	2,840
80	10,000	120	4,440	160	2,500
90	7,900	130	3,790	170	2,210
100	6,400	140	3,270		

3.5—WEB CRIPPLING OF BEAMS

To avoid crippling of flat webs of beams:

(A) Concentrated load located anywhere on the span, or reaction of continuous supports shall not exceed P_{max} in the formula,

(a) .... $P_{max} = t^2 f_b(11.1 + 2.41\sqrt{B/t})$

For any given load or reaction, P, as defined above, the minimum length of bearing, B_{min}, shall be,

(b) .... $*B_{min} = t\left[\dfrac{P}{2.41t^2 f_b} - 4.62\right]^2$

(B) Concentrated loads on the outer ends of cantilevers, or simple end reactions of beams, shall not exceed P_{max} in the formula,

(c) .... $P_{max} = t^2 f_b(7.4 + 0.93\sqrt{B/t})$

For any given load or reaction, P, as defined above, the minimum length of bearing, B_{min}, shall be,

(d) .... $*B_{min} = t\left[\dfrac{P}{0.93t^2 f_b} - 8.00\right]^2$

In these formulas.

P = concentrated load, or reaction, in pounds.

t = web thickness, in inches.

B = length of bearing, in inches.

f_b = basic design stress, in pounds per square inch (Sec. 3.1).

Where a concentrated load is applied to the top flange of a beam at the support (such as a stud resting on a beam over the support), the required length of bearing to provide for the load on the top flange and for the reaction on the bottom flange shall be determined independently for the upper and lower flanges by the above formulas.

Where the web of a beam consists of two or more sheets, each sheet shall be considered as a separate member carrying its share of the load or reaction.

3.6—AXIALLY LOADED COMPRESSION MEMBERS

3.6.1 Unit Stress**

The allowable unit stress, P/A, for axially loaded compression members shall be:

For Grade C Steel†

L/r less than $132/\sqrt{Q}$: $P/A = 15300Q - 0.437Q^2(L/r)^2$

L/r greater than $132/\sqrt{Q}$: $P/A = \dfrac{134{,}000{,}000}{(L/r)^2}$

* In formulas (b) and (d) should the first term within the brackets be smaller than the second, the smallest length of bearing which is practicable will be safe and sufficient, as far as web crippling is concerned, for the given load.

** For continuous compression chords of trusses with rigid welded connections at panel points, tests in accordance with Sec. 6 may show higher allowable load-carrying capacity than calculated by these formulas.

† P/A values for Grade C Steel are given in Chart 4, p. 11.

For Other Grades of Steel

L/r less than $\dfrac{24,000}{\sqrt{f_y}\ \sqrt{Q}}$: $\dfrac{P}{A} = 0.464Qf_y - \dfrac{4.01Q^2(f_y)^2(L/r)^2}{10,000,000,000}$

L/r greater than $\dfrac{24,000}{\sqrt{f_y}\ \sqrt{Q}}$: $\dfrac{P}{A} = \dfrac{134,000,000}{(L/r)^2}$

In the above formulas,

P = total allowable load, pounds;
A = full, unreduced cross-sectional area of the member;
L = unsupported length of member, inches;
r = radius of gyration of full, unreduced cross-section, inches;
f_y = yield point of steel, lb. per sq. in.; and
Q = a factor determined as follows:

(a) For members composed entirely of *stiffened* elements, "Q" is the ratio between the effective design area, as determined from the effective design widths of such elements, and the full or gross area of the cross-section. The effective design area used in determining Q is to be based upon the basic design stress f_b as defined in Sec. 3.1.

(b) For members composed entirely of *unstiffened* elements, "Q" is the ratio between the allowable compression stress f_c for the weakest element of the cross-section (the element having the largest flat-width ratio) and the basic design stress f_b; where f_c is as defined in Sec. 3.2, and f_b is as defined in Sec. 3.1.

(c) For members composed of *both stiffened and unstiffened* elements the factor "Q" is to be the product of a stress factor Q_s computed as outlined in (b) above and an area factor Q_a computed as outlined in (a) above, except that the stress upon which Q_a is to be based shall be that value of the unit stress f_c which is used in computing Q_s; and the effective area to be used in computing Q_a shall include the full area of all unstiffened elements. (See Examples in Part V.)

See Chart 4, p. 11, for values of P/A for Grade C Steel.

3.6.2 Maximum Slenderness Ratio.

The maximum allowable ratio L/r of unsupported length, L, to radius of gyration, r, of compression members shall be as follows:

(a) Columns, and other primary compression members, except as provided otherwise in this Section 120
(b) Load-bearing studs 160
(c) Secondary members 200
(d) During construction 300

If members which are temporarily unbraced during construction are to act as permanent load-carrying members in the completed structure they must be so braced prior to completion of the structure as to reduce the L/r ratio to a value not exceeding that given in (a), (b), or (c) above, whichever may apply.

3.7—COMBINED AXIAL AND BENDING STRESSES

For members subject to both axial compression and bending stresses, the member shall be so proportioned that the quantity—

$\dfrac{f_a}{F_a} + \dfrac{f'_b}{F_b}$ shall not exceed unity, where

F_a = maximum axial unit stress in compression that is permitted by this Specification where axial stress only exists. (Sec. 3.6.1)

F_b = maximum bending unit stress in compression that is permitted by this Specification where bending stress only exists. (Sec. 3.1 and 3.2)

f_a = axial unit stress = axial load divided by full cross-sectional area of member.

f'_b = bending unit stress = bending moment divided by section modulus of member, noting that for members having stiffened compression elements the section modulus shall be based upon the effective design widths of such elements.

SECTION 4. CONNECTIONS

4.1—GENERAL

Connections shall be designed to transmit the maximum stress in the connected member with proper regard for eccentricity. In the case of members subject to reversal of stress the connection shall be proportioned for the sum of the stresses.

4.2—WELDS

4.2.1 Fusion Welds.

For all grades of steel, fusion welds shall be proportioned so that the unit stresses therein shall not exceed 11,300 lbs. per square inch in shear, or 13,000 lbs. per square inch in tension. Stresses due to eccentricity of loading, if any, shall be combined with the primary stresses; and the combined unit stresses shall not exceed the values given above.

4.2.2 Resistance Welds.

In plates or sheets joined by spot welding, the design strength per spot shall be as follows:

Thickness of Thinnest Outside Sheet, Inches	Design Strength per Spot, Pounds
.010	50
.020	125
.030	225
.040	350
.050	525
.060	725
.080	1075
.094	1375
.109	1650
.125	2000
.155	2700
.185	3300

(The above values are those recommended by the American Welding Society in its Recommended Practice for the Spot Welding of Low Carbon Steel. They are applicable for all grades of low carbon steel up to a yield point of 70,000 lb. per sq. in. and are based on a factor of safety of three. The welding procedure shall conform to that set forth in the Recommended Practice published by the American Welding Society, dated August 1944.)

4.3—WELDS CONNECTING TWO CHANNELS TO FORM AN I-SECTION FOR USE AS A BEAM

The required tension strength of welds connecting two channels to form an I-beam shall be determined from the following formula:

$S_w = \dfrac{mqs}{2c}$ where

S_w = required strength of weld in tension, (pounds)

s = longitudinal spacing of welds, (inches)

c = vertical distance between the two rows of welds near or at top and bottom flange, (inches)

q = intensity of load per linear inch of beam, (pounds)
(For method of determination, see below.)

m (for simple channels without stiffening lips at the outer edges)

$= \dfrac{w^2}{2w + h/3}$, the distance of shear center of channel from axis of web

m (for C-shaped channels with stiffening lips at the outer edges)

$= \left(\dfrac{wh}{2}\right) \left[\dfrac{wh + 2d(h - d)}{wh^2 + h^3/6 + d(h - d)^2}\right]$, the distance of shear center of channel from axis of web.

w = projection of flanges beyond web, in inches. (For channels with flanges of unequal width, w shall be taken as the width of the wider flange.)

h = depth of channel or beam, (inches)

d = depth of lip, (inches)

The intensity of load, q, is obtained by dividing the magnitude of concentrated loads or reactions (in pounds) by the length of bearing or by longitudinal spacing of welds, s, (in inches), whichever is larger. For beams designed for "uniformly distributed load," the intensity q shall be taken equal to three times the intensity of the uniformly distributed design load (in pounds per linear inch).

The required strength of welds depends upon the intensity of the load directly at the weld. Therefore, if uniform diameter and spacing of welds are used over the whole length of the beam, the necessary strength of the welds shall be determined at the point of maximum local load intensity. In cases where this procedure would result in uneconomically close spacing either one of the following methods may be adopted: (a) the weld spacing may be varied along the beam according to the variation of the load intensity; or (b) reinforcing cover plates may be welded to the flanges at points where concentrated loads occur. The strength *in shear* of the welds connecting these plates to the flanges shall then be determined from the formula for S_w specified herein but "c" shall then represent the depth of the beam.

SECTION 5. DESIGN OF BRACED WALL STUDS

The safe load-carrying capacity of a stud may be computed on the basis that wall material or sheathing (attached to the stud) furnishes adequate lateral support to the stud in the plane of the wall, provided the wall material and its attachments to the stud comply with the following requirements:

(a) Wall material or sheathing must be attached to both faces or flanges of the stud. (b) The spacing of attachments of wall material to stud shall not exceed "a" as determined from the formula*

$$a = r_2 \sqrt{\frac{29{,}500{,}000}{f_y}}$$

where $r_2 = \sqrt{I_2/A}$ the radius of gyration of the stud corresponding to the moment of inertia I_2 about the principal axis of the stud perpendicular to the wall, in inches.

f_y = minimum specified yield point of steel, lb. per sq. in.

(c) The modulus of elastic support, k, to be exerted laterally by the wall material and its attachments to the stud, shall be not less than —

for Steel of Grade C, $k = \frac{4.5\ a\ A^2}{I_2}$; for Grade B, $k = \frac{3.7\ a\ A^2}{I_2}$

for Steel of Grade A, $k = \frac{2.6\ a\ A^2}{I_2}$; for steel of grade other than Grades A, B, and C: $k = \frac{f_y^2\, a\, A^2}{240{,}000{,}000\ I_2}$

where

f_y = yield point of the steel in the studs, pounds per square inch.

a = spacing of attachments of wall material to stud measured along the length of stud, (a = 1 for continuous attachment), in inches.

A = area of cross section of stud, in square inches.

I_2 = moment of inertia of cross section of stud about its principal axis perpendicular to wall, inches^4.

k = spring constant or modulus of elastic support of wall material (on each [one] side of stud) plus attachment, i.e., $k = F/y$ where F is the force in pounds which produces an elongation of y inches of a strip of wall material of width "a" and of length equal to the distance between adjacent studs.

See Examples in Part V.

Whether or not a given wall material satisfies this "k" requirement shall be established by test procedure as defined in Appendix 4 of the Design Specification. Also included in Appendix 4 of the Design Specification are a number of "k" values as determined by the test procedure described, for several common types of wall sheathing.

(d) The lateral force, F, which each single attachment of the wall material shall be capable of exerting on the stud in the plane of the wall (in order to prevent lateral buckling of the stud) shall not be less than —

* This formula for spacing is based upon the wall material having a minimum k according to (c). In installations where the test value, k, of the wall material exceeds the required minimum value, the spacing can be determined from the actual value of k by using the formula: $a = \frac{8\ EI_2k}{A^2 f_y^2}$. In such cases, the test value of k should be used in formula (d), also. Further, the slenderness ratio, a/r_2, should not exceed $L/2r_1$, where L = length stud, and r_1 = radius gyration about axis parallel to wall.

$$F_{min} = \frac{k_{min}eP}{2\sqrt{EI_2k_{min}/a} - P}$$

where

k_{min} = modulus of elastic support, value to be determined from paragraph (c) of this section.

$e = \frac{\text{stud length in inches}}{240}$

P = design load on stud, in pounds.

I_2 = moment of inertia of stud about its principal axis perpendicular to the wall, in inches^4.

a = spacing of attachments measured along stud, in inches, (a = 1 inch for continuous attachment).

E = Modulus of Elasticity = 29,500,000 lb. per sq. in.

See Examples in Part V.

Whether or not a given means of attachment satisfies this requirement shall be ascertained by test procedure as specified in Appendix 4 of the Design Specification.

SECTION 6. TESTS

6.1—TESTS FOR SPECIAL CASES

Where elements, assemblies, or details of structural members formed from sheet or strip steel are such that calculation of their strength, safe load-carrying capacity, deflection, or other properties cannot be made in accordance with provisions of this Specification, such properties may be determined by suitable tests.

6.2—TEST PROCEDURE

It is recommended that tests for the purposes defined in Sec. 6.1 be conducted in accordance with the following procedure.

(a) Where practicable, evaluation of test results should be made on the basis of the mean values resulting from tests of not less than three identical specimens, provided the deviation of any individual test result from the mean value obtained from all tests does not exceed ± 10%. If such deviation from the mean exceeds 10%, at least three more tests of the same kind should be made. The average of the three lowest values of all tests made should then be regarded as the result of the series of tests.

(b) Determinations of allowable load-carrying capacity should be made on the basis that the member, assembly or connection should be capable of sustaining a total load of twice the design load, and that harmful local distortions should not develop at a total load equal to the design dead load plus one and one-half times the design live load.

(c) Determination of elastic properties should be based on the deformations developed at either (1) 75% of the maximum load which can be sustained or (2) a total load equal to the design dead load plus one and one-half times the design live load, whichever is the larger.

(d) Tests shall be made by an independent testing laboratory or by a manufacturer's testing laboratory.

PART IV

SUPPLEMENTARY INFORMATION

The Linear Method for Computing Properties of Formed Sections

Computation of properties of formed sections may be simplified by using a so-called linear method, in which the material of the section is considered concentrated along the center line of the steel sheet and the area elements replaced by straight or curved "line elements." The thickness dimension, t, is introduced after the linear computations have been completed.

The total area of the section is found from the relation:
Area $= L_t \cdot t$, where L_t is the total length of all line elements.

The moment of inertia of the section, I, is found from the relation:
$I = I' \cdot t$, where I' is the moment of inertia of the center line of the steel sheet. The section modulus is computed as usual by dividing I or $I' \cdot t$ by the distance from the neutral axis to *the extreme fiber,* not to the center line of the extreme element.

First power dimensions, such as $\bar{x}, \bar{y}$ and r (radius of gyration) are obtained directly by the linear method and do not involve the thickness dimension.

When the flat width, w, of a stiffened compression element is reduced for design purposes, the effective design width, b, is used directly to compute the total effective length L_{eff} of the line elements, as shown in Example No. 12, p. 64.

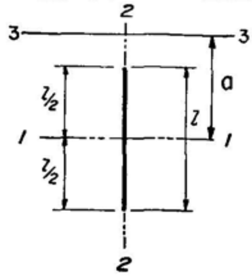
The elements into which most sections may be divided for application of the linear method consist of straight lines and circular arcs. For convenient reference, the moments of inertia and location of centroid of such elements are identified in the sketches and formulas on page 41.

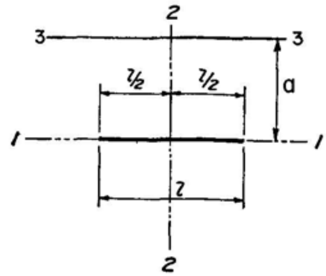
The formulas for line elements are exact, since the line as such has no thickness dimension; but in computing the properties of an actual section, where the line element represents an actual element with a thickness dimension, the results will be approximate for the following reasons:

1. The moment of inertia of a straight actual element about its longitudinal axis is considered negligible.
2. The moment of inertia of a straight (actual) element inclined to the axes of reference is slightly larger than that of the corresponding line element, but for elements of like length the error involved is even less than the error involved in neglecting the moment of inertia of the element about its longitudinal axis. Obviously, the error disappears when the element is normal to the axis.
3. Small errors are involved in using the properties of a linear arc to find those of an actual corner, but with the usual small corner radii the error in the location of the centroid of the corner is of little importance, and the moment of inertia generally negligible. (See Table 11, p. 77, showing *actual* properties of a 90° corner.) When the mean radius of a circular element is over four times its thickness, as for tubular sections and for sheets with circular corrugations, the error in using linear arc properties practically disappears.

Examples 11 to 14, inclusive, illustrating the application of the linear method, appear on pp. 63 to 67.

PROPERTIES OF LINE ELEMENTS



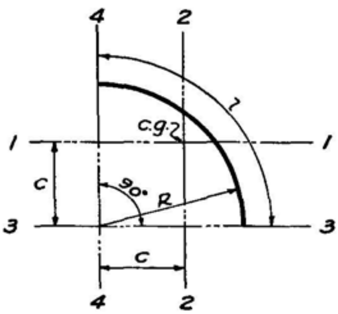


$$I_1 = \frac{l^3}{12} \qquad I_2 = 0$$

$$I_3 = la^2 + \frac{l^3}{12} = l\left(a^2 + \frac{l^2}{12}\right)$$

$$I_1 = 0 \qquad I_2 = \frac{l^3}{12}$$

$$I_3 = la^2$$

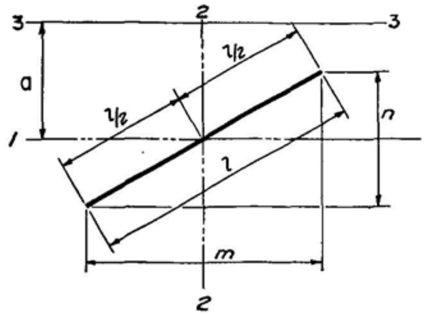


$$l = 1.57 \ R$$

$$c = 0.637 \ R$$

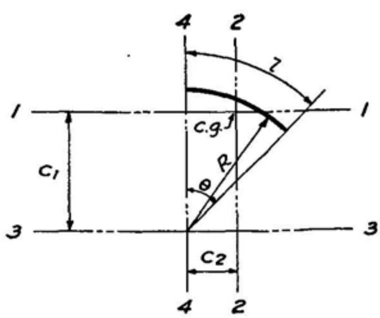
$$I_1 = I_2 = 0.149 \ R^3$$

$$I_3 = I_4 = 0.785 \ R^3$$



$$I_1 = \frac{ln^2}{12} \qquad I_2 = \frac{lm^2}{12}$$

$$I_3 = la^2 + \frac{ln^2}{12} = l\left(a^2 + \frac{n^2}{12}\right)$$



θ (expressed in radians) $= 0.01745 \times \theta$ (expressed in degrees and decimals thereof).

$$l = \theta R$$

$$c_1 = \frac{R \sin \theta}{\theta} \qquad c_2 = \frac{R(1 - \cos \theta)}{\theta}$$

$$I_1 = \left[\frac{\theta + (\sin\theta)(\cos\theta)}{2} - \frac{(\sin\theta)^2}{\theta}\right] R^3 \qquad I_3 = \left[\frac{\theta + (\sin\theta)(\cos\theta)}{2}\right] R^3$$

$$I_2 = \left[\frac{\theta - (\sin\theta)(\cos\theta)}{2} - \frac{(1 - \cos\theta)^2}{\theta}\right] R^3 \qquad I_4 = \left[\frac{\theta - (\sin\theta)(\cos\theta)}{2}\right] R^3$$

RIBBED STEEL ROOF DECK

The types of ribbed steel roof deck commonly used have a decidedly unsymmetrical cross-section. They consist of a flat deck (wide compression flange) with shallow longitudinal ribs spaced 6 inches, or less, center to center. The allowable basic design stress limits the stress in tension. The actual unit stress in compression, however, governs the effective width and thus the structural properties of the cross section of the deck; hence, that stress, which is less than the tension or allowable basic design stress, can be determined only by successive trials using an effective width corresponding to an assumed unit compression stress and calculating the stress to determine whether it agrees with that assumed. That procedure of "cut and try" is cumbersome.

Accordingly, applicable only to ribbed steel roof decks having shallow longitudinal ribs spaced not over 6 inches center to center, and bearing on end supports spaced not over 10 ft. apart, the simplified design procedure described below has been evolved.

Although the actual cross sectional dimensions of the several types of ribbed steel roof deck produced by different manufacturers vary somewhat, it was found that design procedure could be simplified, within the tolerance of accuracy (about 5%) generally allowed in engineering practice, by limiting the effective design width of the wide, top (compression) flange between ribs to the values tabulated below:

Thickness of Deck Metal	Width of Top Flange Effective
#18 U. S. Standard Gage	3/4 of the clear, flat width of flange
#20 U. S. Standard Gage	5/8 of the clear, flat width of flange
#22 U. S. Standard Gage	1/2 of the clear, flat width of flange

The results obtained by using the values tabulated above, have been checked against those obtained by rigorous analysis, and have been found to be in reasonably close agreement. Use of the values tabulated above eliminates the cut and try procedure required by a rigorous analysis, and greatly simplifies the design, or checking of designs, of ribbed steel roof deck.

In some instances the effective width, as determined from a rigorous analysis of a standard type of ribbed deck, may differ from the values given in the preceding tabulation by more than 5% but the load or deflection values, as obtained from the effective widths of the tabulation, will differ from the true value by a much smaller percentage; this is because of the unbalanced cross section of steel roof decks. Since the compression flange is much wider than the tension flange, a decrease of effective width causes the moment of inertia to decrease, but it also causes the distance from the neutral axis to the bottom flange (the extreme fiber) to decrease; thus the two changes tend to offset each other, and there is little net change in the section modulus.

For the reason given above, the effective width values given in the tabulation are not proper for application to sections other than steel roof decks that have unbalanced cross section with shallow longitudinal ribs spaced not over 6 inches center to center.

PROPORTIONS OF SYMMETRICAL CHANNELS AND ZEES FOR MAXIMUM EFFECTIVENESS

The following approximate general relationships may be of value to the designer in selecting the most economical channel or zee section to resist bending about the X-X axis.

Sections with Unstiffened Flanges. For a given depth/thickness ratio, h/t, and a given basic unit stress, f_b, there is a definite flat width ratio for the flanges, $(w/t)_M$, which yields a maximum resisting moment for the section. This optimum flat-width ratio of the flanges is given by the following approximate formula:

$(w/t)_M = 12$ or $= A - (1/12)\ (h/t)$, whichever is the larger where

$A = \dfrac{15f_b - 48870}{f_b - 8150}$. Values of A for different values of f_b follow:

$$\begin{aligned} A &= 29 \quad \text{for } f_b = 13{,}500 \text{ p.s.i.} \\ &= 24 \qquad\qquad = 16{,}500 \\ &= 22 \qquad\qquad = 18{,}000 \\ &= 19 \qquad\qquad = 27{,}000 \end{aligned}$$

The maximum efficiency of a channel or zee section with unstiffened flanges, in bending about the X-X axis, as measured by resisting moment per unit of weight, occurs when the flat-width ratio w/t of the flanges is equal to 12.

Sections with Stiffened Flanges. For a given depth/thickness ratio, h/t, the resisting moment about the X-X axis increases as the flat-width ratio, w/t, of the flanges increases, and is a maximum when that flat-width ratio reaches the limiting value of 60. The proportions for maximum efficiency (maximum resisting moment per unit of weight) of sections having stiffened flanges are not as clearly defined as for those having unstiffened flanges. There are, however, a number of general relationships which may be summarized as follows:

(1) Efficiency increases as w/t increases from zero to 25.

(2) Efficiency decreases as w/t increases from 25 to 60, provided h/t is less than $(h/t)_L$, where $(h/t)_L$ is as follows:

$$\begin{aligned} (h/t)_L &= 80 \quad \text{for } f_b = 13{,}500 \text{ p.s.i.} \\ &= 85 \qquad\qquad = 16{,}500 \\ &= 90 \qquad\qquad = 18{,}000 \\ &= 133 \qquad\quad\ = 27{,}000 \end{aligned}$$

(3) When h/t is greater than $(h/t)_L$, efficiency increases as w/t increases from 25 to about 47.5, and then decreases as w/t increases from 47.5 to 60.

BOLTED CONNECTIONS

There is some evidence that the bearing stress computations commonly used in the design of bolted structural connections are not entirely applicable to connections in which the thickness of the connected parts is very small compared to the bolt diameter. Studies and tests pointing to the development of a rational design procedure for the use of bolted connections in thin metal are in process of development. One series of tests, carried out at the University of Michigan on sheet thicknesses from No. 8 gage (0.1644″) to No. 16 gage (0.0598″) with bolts from 1/2″ to 1-1/4″ in diameter, appears to indicate that conventional shear computations are valid but that conventional bearing stress determinations might be replaced by the following formulas:

(a) For a single row of bolts perpendicular to the direction of force:

$$P = f_b(t)(e)(n)$$

(b) For bolts arranged in rows parallel to the direction of force:

$$P = f_b\, t\,[e + (n - 1)(s - D)]$$

where

P = safe loads in pounds, that will avoid excessive deformations of bolt holes.

e = distance in inches measured in direction of force from edge of sheet to center of hole nearest edge, except that where e exceeds 2½ inches or 3½ times the bolt diameter substitute either 2½ inches or 3½ D, whichever is smaller, for e.

s = spacing of bolts in inches in direction of the force, except that where s exceeds 2½ inches or 3½ times the bolt diameter substitute either 2½ inches or 3½ D, whichever is smaller, for s.

f_b = the basic design stress

t = thickness of sheet, inches

n = number of bolts

D = bolt diameter, inches.

Bolted connections also must be checked for shear of bolts and for tension on the net section, following conventional design procedure.

WALL-BRACED STUDS*

It is standard practice in the design of wood-stud framing to take cognizance of the lateral support afforded by the collateral materials attached to those studs to form partition or wall panels.

Wall-braced panels in which the columns or studs are cold-formed, light weight shapes of sheet or strip steel are now finding increasingly wide application for structural purposes. In such panels the collateral wall material also provides the steel column or stud with comparable lateral support in the plane of the wall. Evaluation of the bracing abilities of the collateral wall materials and their attachments, as well as of the amount of support necessary to prevent failure of the steel studs in the plane of the wall, becomes essential for sound design procedure.

Such an evaluation appears in Sec. 5, Design Specification, p. 37. Proper design criteria for the necessary qualifications of collateral materials to serve as lateral bracing of studs are established for the first time. (See Example No. 9, p. 60.)

Wall materials generally are selected for properties other than their ability to provide lateral support to the stud members to which they are attached. For that reason, they generally are capable of providing a degree of elastic support far in excess of the minimum requirement for bracing the stud. (Sec. 5, Design Specification, p. 37.) Hence, it is usually possible to increase the spacing of attachments beyond that of Formula (b)** of Sec. 5, which is based on the minimum amount of lateral support necessary to brace the stud.

Cornell Bulletin 35/2 supplements Sec. 5 of the Design Specification with

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* Bulletin 35/2 of Cornell University Engineering Experiment Station, from which the first two paragraphs above are quoted, gives details of the investigation through which the design criteria, herein discussed, were established.

** The formula of the Design Specification:

$a = r_2 \sqrt{E/f_y}$ is based upon the minimum modulus of elastic support necessary to brace the stud.

a general formula for the allowable spacing of wall attachments along the stud member, taking cognizance of the *actual* modulus of elastic support, k, of the wall material and attachments used, as determined by tests; that formula is:

$$a = \frac{8\ E\ I_2\ k}{A^2\ f_y^2} \text{ but must not exceed } \frac{r_2 L}{2 r_1} \text{ where}$$

a = spacing of attachments of wall material to stud measured along the length of stud, ($a = 1$ for continuous attachment), in inches.
E = modulus of elasticity of stud material, (29,500,000 for steel).
I_2 = moment of inertia of stud about axis perpendicular to the wall.
k = modulus of elastic support as actually provided by one single attachment (one side of stud). For test method of determining k see Appendix 4, Design Specification, April 1946.
A = cross-section area of stud (sq. in.).
f_y = yield point of stud material.
r_1 = radius of gyration of stud about the axis parallel to wall.
r_2 = radius of gyration of stud about the axis perpendicular to wall.
L = Length of stud.

If the above formula for spacing "a" is used in place of the more restrictive stipulation Sec. 5. b of the Design Specification, the actual test value of "k", as defined above, must be substituted for "k_{min}" in the formula for "F_{min}" (the required strength of attachment) in Sec. 5. d, Design Specification, p. 39.

The effectiveness of the attachments usually is the critical design factor affecting the lateral bracing qualifications of the wall material.

THICKNESS LIMITATIONS

In the design of light steel members, proper proportioning of the component parts, consistent with their width to thickness ratio and their unit stress, is the factor of importance to assure safe structural performance; the thickness of the metal itself is, per se, not a critical factor. Members formed of extremely thin steel will function satisfactorily, if designed in accordance with the procedure prescribed in the Design Specification and in this Manual.

No minimum thickness limitations are necessary to assure sound structural behavior. In building construction, however, it is not unusual to specify certain minimum thickness limitations based upon prevailing practices, practical considerations, and experience in handling standard products in the field. The following provision for light steel constructions used within buildings, based on Bulletin V, Steel Regulations, published by the American Iron and Steel Institute in January, 1947, is typical of sound regulatory procedure:

Thickness: Steel of Grade C, used to form individual structural (load-carrying) members, shall be of thickness not less than 18 U.S. Standard Gage, provided that the use of material of less thickness may be allowed upon the submission of test data from approved authorities verifying the structural behavior of the members formed from such material.

Exception: Steel used to form load-carrying panels, including ribbed steel roof deck construction (see p. 42) shall be of thickness not less than 22 U.S. Standard Gage.

PROTECTION (PAINT)

Structural use of light gage steel members covers a long record of service in many types of machines, mechanical equipment, airplanes and building construction. In some of these uses the structural members have been subjected to severe conditions of exposure, shock, vibration and sudden repetitions of load, far more severe than the conditions usually prevailing in buildings.

A building constructed as long ago as 1850 with lightweight sheet iron joists, (1/16 and 1/8 inch thick) showed no apparent deterioration of structural members when the building was taken down to make way for a modern structure, after standing for over 50 years.

Standard specifications require the application of a protective paint coating to light structural members and constructions before they are shipped from the manufacturing or fabricating plant. Usually it is not practicable to paint these light steel secondary constructions after their installation, nor is it standard practice to require them to be painted after erection.

In 1940-42 the Pittsburgh Testing Laboratory conducted a survey to determine whether these painting practices have provided effective protection to light steel members. The report of the laboratory shows that the 50 installations inspected, covering locations where climatic conditions were most severe, were generally in excellent condition and structurally sound, and "appear to justify the conclusion that the commercial protective paint coatings applied to light steel structural members of the types included in this survey provide effective protection to the steel in actual service and that such light steel structural members may be expected to retain their structural properties during the life of a building when enclosed within the confines of a building." Details of the report are given in the booklet, "The Durability of Lightweight Types of Steel Construction," published by the American Iron and Steel Institute. Copies are available upon request.

A study made by the Engineering Research Institute of the University of Michigan confirms the above conclusion. Details of that investigation appear in the University's Bulletin No. 30, "Durability of Light Weight Steel Construction," of which there are three parts: Part I, Effect of Copper and Other Alloys Upon Atmospheric Corrosion of Steel; Part II, A Study of the Service Records of Light Weight Steel Construction; Part III, Protection of Steel Surfaces from Atmospheric Corrosion.

PART V

EXAMPLES

The following examples are intended to illustrate the application of various provisions of the Design Specifications, and use of the Tables of Properties and other data given in this Manual.

EXAMPLE No. 1

Beam strength and deflection (using Tables 1 and 2).

STEEL: Grade C ($f_b = 18{,}000$ p.s.i.)
SPAN: 12′-0″; beam continuously braced along top flanges.

LIVE LOAD:	120 lbs. per lin. ft.
DEAD LOAD, incl. weight of beam	40 lbs. per lin. ft.
TOTAL LOAD:	160 lbs. per lin. ft.

Deflection due to live load: not to exceed 1/360 of span or 0.40 in.

REQUIRED: A section as shallow as the deflection limitation will permit.

(a) Beam strength requirement:

$$\text{Moment } M = \frac{1}{8} \times 160 \times 12.0^2 \times 12 = 34560 \text{ in.-lbs.}$$

$$S_{req} = \frac{M}{f_b} = \frac{34560}{18000} = 1.92 \text{ in.}^3$$

Try 5 x 4 double channel section with stiffened flanges of 14 ga. steel (Table No. 1), $S_x = 2.24$ in.^{3}, $I_x = 5.60$

(b) Deflection requirement:

$$\text{Live Load Moment } M = \frac{1}{8} \times 120 \times 12.0^2 \times 12 = 25920 \text{ in.-lbs.}$$

$$\text{Deflection: } \frac{5WL^3}{384EI_x} = \frac{5ML^2}{48EI_x} = \frac{5 \times 25920 \times (12.0 \times 12.0)^2}{48 \times 29{,}500{,}000 \times 5.6} = 0.34 \text{ in.}$$

This section is satisfactory, since the deflection is less than 0.40 in.

If the section selected has a flange with w/t ratio over 25, I_x would vary with the unit stress (and with M); hence, it would have been necessary to compute the deflection for total load and deduct from it the deflection for dead load only, using appropriate values for I_x in each instance, as calculated from the expression for I_x given in Table No. 1.

NOTE: If a section with unstiffened flanges is to be used, then a 7 x 3 x 14 ga. section (Table No. 2) with $S'_x = 2.18$ in.^{3} and $I_x = 9.08$ in.^{4} will obviously satisfy both the strength and the deflection requirements, as would a 6 x 3 x 12 ga. section.

EXAMPLE No. 2 

Spacing of lateral bracing and length of end bearing (using Tables 1 and 2).

GIVEN: For support of the load, the required section modulus = 0.667 in.^{3}

REQUIRED: (1) Selection of proper section from Table 1 or Table 2.
(2) Maximum spacing of lateral bracing.
(3) Minimum length of end bearing to avoid web crippling.

STEEL: Grade C (f_b = 18,000 p.s.i.)

Sections in Table 1 of nominal dimensions from 12″ x 7″ x No. 10 ga. to 3″ x 3½″ x No. 16 ga. inclusive will meet the section modulus (beam strength) requirement, providing the compression flanges are braced laterally at sufficiently close intervals. The maximum allowable spacing of lateral bracing for any of these sections can be determined from the equation given in Sec. 3.3 of the Design Specification, in Part III, by solving for L after substituting for f'_c its value $f_b\left(\dfrac{\text{Required Section Modulus}}{\text{Section Modulus of Member}}\right)$;

thus, $L = r_y \sqrt{\dfrac{250{,}000{,}000}{f'_c}}$ becomes

$$L = r_y \sqrt{\frac{250{,}000{,}000 \times S_x}{\text{Required Section Modulus} \times f_b}} = r_y \sqrt{\frac{250{,}000{,}000 \times S_x}{0.667 \times 18{,}000}}$$

(Values of S_x and r_y are taken from Table 1 for the section selected.) Sections in Table 2 of nominal dimensions from 8″ x 4″ x No. 10 ga. to 4″ x 2¼″ x No. 16 ga. inclusive as well as 3″ x 2¼″ x No. 12 ga. will meet the section modulus (beam strength) requirement providing the compression flanges are braced laterally at sufficiently close intervals. The maximum allowable spacing of lateral bracing for any of these sections can be determined from the equation given in Sec. 3.3 of the Design Specification, by solving for L after substituting for f'_c its value $Kf_b\left(\dfrac{\text{Required Section Modulus}}{\text{Section Modulus of Member}}\right)$. The resulting equation is:

$$L = r_y \sqrt{\frac{250{,}000{,}000 \times S'_x}{\text{Required Section Modulus} \times K \times f_b}}$$

$$= r_y \sqrt{\frac{250{,}000{,}000 \times S'_x}{0.667 \times K \times 18{,}000}}$$

(Values of S'_x, r_y and K are taken from Table 2 for the section selected.)

The minimum length of end bearing required to prevent web crippling can be determined from the equation given in Sec. 3.5 (d) of the Design Specification in Part III.

$$B_{min} = t\left[\frac{\text{Reaction}}{0.93t^2\ f_b} - 8.00\right]^2$$

$$= t\left[\frac{\text{Reaction}}{0.93t^2\ 18{,}000} - 8.00\right]^2$$

(Values of t, the web thickness, are selected from the Table for the section selected.)

Since the web of the member under investigation consists of two sheets, each sheet must be considered as a separate member carrying its share of the load or end reaction. This is effected by using half the value of the reaction in the formulas above.

Should the first term within the brackets of the above equation be smaller than the second term, the shortest length of end bearing that is necessary for proper transfer of the load to the bearing surface will be safe and sufficient for the load being investigated, i.e., web-crippling is not a critical factor in the determination.

EXAMPLE No. 3

Beam strength and deflection (using Table 3).

REQUIRED: (1) Beam (Table 3) of adequate strength, for $f_b = 18{,}000$ p.s.i.

(2) Maximum deflection under live load shall not exceed L/240 or 0.30 in.

STEEL: Grade C ($f_b = 18{,}000$ p.s.i.)

SPAN: L = 6 ft., section continuously braced laterally.

LOAD: Uniformly distributed, w = 120 lb. per lineal foot (live load).

$$\text{APPLIED MOMENT} = \frac{120 \times 6^2 \times 12}{8} = 6480 \text{ in.-lb.}$$

If the angles are to be used with their joined legs turned up, sections in Table 3 having the following nominal dimensions will satisfy the beam strength requirement for the designated live load:

4″ x 4″ x No. 10 ga.
3″ x 3″ x No. 10 ga.
2½″ x 2½″ x No. 10 ga.

If the angles are to be used with their joined legs turned down, sections with the following nominal dimensions also will satisfy the beam strength requirement for the designated live load:

3″ x 3″ x No. 12 ga.
2½″ x 2½″ x No. 12 ga.

However, the beam strength must provide for the dead weight of the section itself, as well as the live load. Investigating that feature, as per **Example 2,** confirms that all these sections satisfy the total beam strength requirement, except the 2½″ x 2½″ x No. 12 ga.

Selecting a section of nominal dimensions 3″ x 3″ x No. 12 ga., with joined legs turned down, and investigating that section (from Table 3, weight = 4.32 lbs. per ft., $I_x = 1.172$ in.^{4}) for deflection:

$$\text{APPLIED MOMENT} = \frac{(120 + 4.32) \times 6^2 \times 12}{8} = 6713 \text{ in.-lb.}$$

$$\text{Maximum deflection} = \frac{5\,WL^3}{384\,EI} = \frac{5\,(124.32 \times 6)\,(6 \times 12)^3}{384 \times 29500000 \times 1.172}$$

$$= 0.105 \text{ inches.}$$

This deflection under total load is less than the maximum deflection, 0.30 in., allowable under live load only, so the section is therefore satisfactory for use under the conditions imposed.

EXAMPLE No. 4

Beam strength and web shear (using Tables 4 or 5).

REQUIRED: (1) Beam (Table 4 or Table 5) of adequate strength, for $f_b = 27{,}000$ p.s.i. (No deflection limitation.)
(2) Check the web strength in shear.

STEEL: Specified Minimum Yield, 50,000 p.s.i. ($f_b = 27{,}000$ p.s.i.)

SPAN: L = 15 ft. = 180 in. Section continuously braced laterally.

LOAD: Two concentrated loads, P = 900 lb., each, applied 60 in. from end supports.

$$\text{Required Section Modulus} = \frac{\text{Maximum Moment}}{f_b}$$

$$= \frac{900 \times 60}{27{,}000}$$

$$= 2.00 \text{ in.}^3 \text{ (for applied loads only)}$$

Both channels and zees (stiffened flanges) in Table 4, of nominal dimensions as follows, will meet the beam strength requirement for the designated loads:

From 12″ x 3½″ x No. 10 ga. to 8″ x 3″ x No. 14 ga.
7″ x 2¾″ x No. 10 ga., No. 12 ga. and No. 14 ga.
6″ x 2½″ x No. 10 ga. and No. 12 ga.

Both channels and zees (unstiffened flanges) in Table 5, of nominal dimensions as listed below, will meet the beam strength requirements for the designated loads:

8″ x 2″ x No. 10 ga. and No. 12 ga.
7″ x 1½″ x No. 10 ga.

Of these sections, the 7″ x 2¾″ x No. 14 channel or zee in Table 4, with a *Section Modulus* S_x, of 2.04 *in.*^{3}, has little margin of beam strength beyond that required for the applied loads. It therefore should be re-examined, including the weight of the section itself, to establish the total beam strength requirement, as follows:

This channel or zee weighs 3.50 lb. per lineal foot and the moment due to this weight alone is: $M = \frac{3.5 \times 15^2 \times 12}{8} = 1181$ in.-lb. The increment of section modulus required to support the weight of the section itself over a span of 15 ft. equals $\frac{1181}{27.000}$ or 0.044 in.^{3}. Therefore the section modulus required to satisfy the total dead and applied load requirement will be 2.00 plus 0.04 or 2.04 in.^{3} This is exactly the section modulus tabulated for this channel or zee; therefore, these sections may be included among those meeting the specified load conditions.

Selecting this same channel or zee (7″ x 2¾″ x No. 14 ga.) for investigation of the shear strength of the web: (Sec. 3.4, Design Specification, in Part III.)

Maximum average shear stress $= \frac{64.000.000}{(h/t)^2}$ but shall not exceed $\frac{2f_b}{3}$

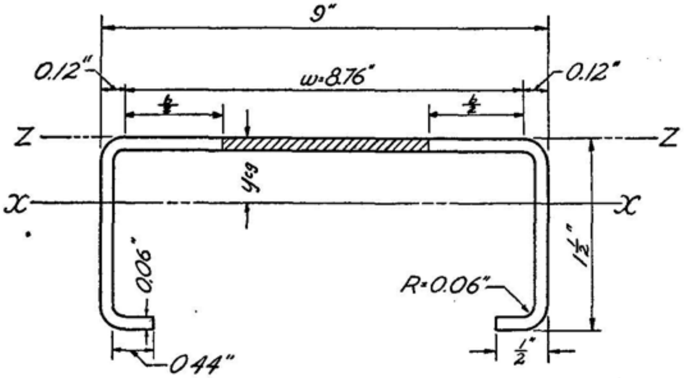
$= \frac{64,000,000}{(6.85/.075)^2}$ but not more than $\frac{2}{3}$ (27,000) or 18,000 p.s.i.

= 7670 p.s.i., which is less than 18,000 p.s.i.

Total shear strength of web = 6.85 x .075 x 7670 = 3940 lb.

This is substantially greater than the end reaction of the beam, 926 lb.; therefore this channel or zee is suitable for use.

EXAMPLE No. 5



Beam strength (using Chart 3A, p. 7).

Steel: Grade "C"
To find: Resisting moment, *When Stress Governs.*

For an Approximation Use an Effective Width of the Top Flange Based on f = 18000 p.s.i.

$\frac{w}{t} = \frac{8.76}{0.06} = 146$

From Chart 3A, p. 7, the effective design width, b, exclusive of corners = 50.0t = 3.00 in.

Total effective width $= 3.00 + 2(0.06) = 3.12$ in., web to web.

Cross-sectional properties (assuming square corners):

Element	Area = F (in.²)	y (in.) (Dist. from top fiber)	(F)y (in.³)	(F)y² (in.⁴)
Top Flange	$3.12 \times 0.06 = 0.1872$	0.03	0.0056	0.0002
Webs	$2 \times 1.50 \times 0.06 = 0.1800$	0.75	0.1350	0.1012
Bottom Flanges	$2 \times 0.44 \times 0.06 = 0.0528$	1.47	0.0776	0.1141
Summation	0.4200		0.2182	0.2155

$$+ I_{cg}(\text{of webs})^{*} = \frac{2 \times 0.06 \times (1.5)^3}{12} = 0.0338$$

$$\overline{0.2493} = I_z$$

$$-0.4200(0.519)^2 = 0.1131$$

$$\text{Moment of Inertia: } I_x = 0.1362$$

Distance of neutral axis from top fiber: $y_{cg} = \dfrac{0.2182}{0.4200} = 0.519$ in.

Section Modulus: $S = \dfrac{0.1362}{1.500 - 0.519} = \dfrac{0.1362}{0.981} = 0.139$ in.^{3}

Resisting Moment: $M = 18{,}000(0.139) = 2500$ in. lb.

Due to the eccentric position of the neutral axis the compressive stress in this section is considerably smaller than the tensile stress. When the tensile stress in the bottom flange is equal to the working stress, 18000 p.s.i., the corresponding compressive stress in the top flange is easily found if the position of the neutral axis is known. This position, however, depends on the effective width of the top flange, which in turn depends on the stress in that flange. It is therefore desirable to locate the neutral axis accurately by successive approximations.

2nd Approximation:

Guided by the determination in the first approximation:

Assume $y = 0.45$ in.

Then the compressive stress in the top flange:

$$f_c = \frac{18000(0.45)}{1.05} = 7710 \text{ p.s.i.}$$

---

* Moments of Inertia of top and bottom flanges about their own centroidal axes have been neglected.

From Chart 3A, p. 7, for this f_c and $w/t = 146$:

$b/t = 71.3$, thus the effective design width, $b = 71.3(0.06) = 4.28$ **in.**

Total effective width $= 4.28 + 2(0.06) = 4.40$ in.

Check assumed position of the neutral axis (assuming square corners):

Element	Area = F (in.²)	y (in.) (Dist. from top fiber)	(F)y (in.³)
Top Flange	4.40 × 0.06 = 0.2640	0.03	0.0079
Webs	2 × 1.50 × 0.06 = 0.1800	0.75	0.1350
Bottom Flanges	2 × 0.44 × 0.06 = 0.0528	1.47	0.0776
Summation	0.4968		0.2205

Distance of axis from top fiber: $y_{cg} = \dfrac{0.2205}{0.4968} = 0.443$ in.

3rd Approximation:

On the basis of the determination above assume $y = 0.443$ in.

$$f_c = \frac{18000(0.443)}{1.057} = 7545 \text{ p.s.i.}$$

From Chart 3A, p. 7, for this f_c and $w/t = 146$:

$b/t = 7.5$, thus the effective design width, $b = 71.5\ (0.06) = 4.29$ in.

Total effective width $= 4.29 + 2(0.06) = 4.41$ in.

Cross-sectional properties (assuming square corners):

Element	Area = F (in.²)	y (in.) (Dist. from top fiber)	(F)y (in.³)	(F)y² (in.⁴)
Top Flange	4.41 × 0.06 = 0.2646	0.03	0.0079	0.0002
Webs	2 × 1.50 × 0.06 = 0.1800	0.75	0.1350	0.1012
Bottom Flanges	2 × 0.44 × 0.06 = 0.0528	1.47	0.0776	0.1141
Summation	0.4974		0.2205	0.2155

$$+ I_{cg}(\text{of webs})^* = \frac{2 \times 0.06 \times (1.5)^3}{12} = 0.0338$$

$$0.2493 = I_x$$

$$-0.4974(0.443)^2 = 0.0976$$

$$\text{Moment of Inertia: } I_x = 0.1517$$

Distance of axis from top fiber:

$$y_{cg} = \frac{0.2205}{0.4974} = 0.443 \text{ in. (checks assumed value)}$$

Section Modulus: $S = \dfrac{0.1517}{1.500 - 0.443} = \dfrac{0.1517}{1.057} = 0.144 \text{ in.}^3$

* Moments of Inertia of top and bottom flanges about their own centroidal axes have been neglected.

Resisting Moment: $M = 18{,}000(0.144) = 2590$ in. lb.

A comparison of these values with the results of the first approximation shows that the final accurate determination allows an increase in design load of about 3.6% (if stress, not deflection, governs) as compared with the values obtained in the simpler, but approximate, determination based on a stress of 18000 lb. per sq. in. in the compression flange.

EXAMPLE No. 6

Deflection (using Chart 3C, p. 9).

STEEL: Grade "C"

TO FIND: Moment of Inertia for *Computing Deflection*

Same Section as Example No. 5, loaded to its capacity Resisting Moment. Since the top flange stress, from Example No. 5, is 7545 p.s.i., that stress will be used for a first approximate computation of the effective width of top flange to be used for deflection determinations.

$w/t = 146$ (as in Example No. 5)

From Chart 3C, p. 9, the effective design width, b, exclusive of corners $= 89.6t = 5.38$ in.

Total effective width $= 5.38 + 2(0.06) = 5.50$ in. web to web.

Properties, assuming square corners, are:

Moment of Inertia: $I = 0.161$ in.^{4}

Distance of neutral axis from top fiber: $y = 0.396$ in.

$$\text{Stress in top flange: } f_c = \frac{2590(0.396)}{0.161} = 6370 \text{ p.s.i.}$$

2nd Approximation:

Guided by the determinations of the first approximation, now assume that the compressive stress, f_c, in the top flange is 5900 p.s.i.

From Chart 3C, p. 9, b/t is determined:

$$\frac{b}{t} = 97.0,$$

whence the effective design width, $b = 97.0\ (0.06) = 5.82$ in.

Total effective width $= 5.82 + 2(0.06) = 5.94$ in.

Properties, assuming square corners, are:

Moment of Inertia: $I_x = 0.1648$ in.^{4}

Distance of neutral axis from top fiber: $y = 0.379$ in.

$$\text{Stress in top flange: } f_c = \frac{2590\,(0.379)}{0.1648} = 5960 \text{ p.s.i.}$$

3rd Approximation:

On the basis of the determination above, assume $f_c = 5980$ p.s.i.

From the formula of Sec. 2.3.1, b/t is now calculated:

$$\frac{b}{t} = \frac{10{,}320}{\sqrt{5980}}\left(1 - \frac{3120}{146\sqrt{5980}}\right) = 96.6,$$ (This value could be taken from Chart 3C.)

whence the effective design width, b, $= 96.6(0.06) = 5.80$ in.

Total effective width $= 5.80 + 2(0.06) = 5.92$ in.

Cross-sectional properties (assuming square corners):

Element	Area = F (in.)^2	y (in.) (Dist. from top fiber)	(F)y (in.^{3})	(F)y^2 (in.^{4})
Top Flange	$5.92 \times 0.06 = 0.3552$	0.03	0.0107	0.0003
Webs	$2 \times 1.50 \times 0.06 = 0.1800$	0.75	0.1350	0.1012
Bottom Flanges	$2 \times 0.44 \times 0.06 = 0.0528$	1.47	0.0776	0.1141
Summation	0.5880		0.2233	0.2156

$$+ I_{cg} \text{ (of webs)*} = \frac{2 \times 0.06 \times (1.5)^3}{12} = 0.0338$$

$$0.2494 = I_z$$

Distance of neutral axis from top fiber:

$$y_{cg} = \frac{0.2233}{0.5880} = 0.380 \text{ in.}$$

$$-0.5880(0.380)^2 = \quad -0.0848$$

$$\text{Moment of Inertia: } I_x = 0.1646$$

$$\text{Stress in top flange} = \frac{2590(0.380)}{0.1646} = 5980 \text{ p.s.i. (checks assumed stress)}$$

A comparison of the value of 0.1646 in.^{4} for the moment of inertia, I, (for deflection) with the value of 0.1517 in.^{4} which was used in the stress calculations of Example 5 shows that the deflection, as determined, will actually be approximately 8% less than if computed on the basis of the same moment of inertia used in the stress calculations. It will also be noted that the first approximation of the deflection determinations gave a value of I which was within approximately 2% of the correct value.

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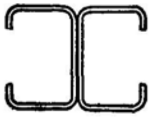
* Moments of Inertia of top and bottom flanges about their own centroidal axes have been neglected.

EXAMPLE No. 7



Axially loaded compression member (using Table No. 1 and Chart 4, p. 11).

REQUIRED: A column (Table 1) or compression member of adequate load capacity, for the basic stress $f_b = 18{,}000$ p.s.i., to be used as a primary support.

STEEL: Grade C ($f_b = 18{,}000$ p.s.i.)

UNSUPPORTED LENGTH, L: 12 ft. 6 in. = 150 in.

AXIAL LOAD: P = 30,000 lb.

For a column or primary support, L/r must not exceed 120 (Sec. 3.6.2, Design Specification, in Part III). Therefore the allowable unit stress P/A for the member under investigation is: $P/A = 15{,}300\ Q - 0.437Q^2\ (L/r)^2$, according to Sec. 3.6.1, Design Specification.

Since r_y is less than r_x for all the double channel (stiffened) sections in Table 1, r_y will govern. Since r_y must equal at least $\frac{150}{120}$ or 1.25 in., the section, according to Table 1, must be among those of nominal dimensions from 12″ x 7″ x No. 10 ga. to 7″ x 5½″ x No. 16 ga. inclusive. For these sections r_y ranges from 1.26 in. min. to 1.60 in. max.

Assuming that the web-to-web connections of the double channels are relatively closely spaced, the maximum values tabulated for Q will apply. For all the sections cited above, Q_{max} ranges from 0.780 to 1.000 inclusive.

The allowable unit stress, P/A, therefore will not be greater than $[15{,}300 \times 1.00] - [0.437 \times 1.00^2 \times (150/1.60)^2]$, or 11,470 p.s.i., and will not be less than $[15{,}300 \times 0.780] - [0.437 \times 0.780^2 \times (150/1.26)^2]$, or 8180 p.s.i. (These values can be obtained directly from Chart 4, p. 11.) The average of these values, 9820 p.s.i., may be used for tentatively selecting a section which must then be tested by application of the formula: allowable unit stress $= [15{,}300 \times Q] - [0.437Q^2 (L/r)^2]$.

Assuming P/A value of 9820 p.s.i. for tentative selection:

$$\text{Area of section, } A = \frac{30{,}000}{9820} \text{ (approx.)} = 3.06 \text{ sq. in. (approx.)}$$

The section whose area most nearly approximates 3.06 sq. in. is that double channel section of nominal dimensions 8″ x 6″ x No. 12 ga., with an area = 3.10 sq. in.; Q = 0.963; $r_y = 1.38$.

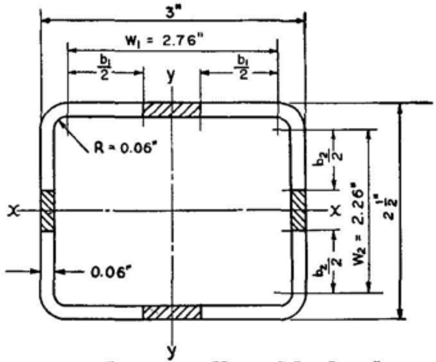
Testing this section:

From Chart 4, p. 11, with Q = 0.963 and L/r = 150/1.38 = 108.7, the corresponding P/A value is 9950 p.s.i.

Allowable axial load = Area x Allowable unit stress
= 3.10 x 9950
= 30,845 lb.

The section tentatively selected is therefore suitable for use. Other sections falling either side of this section in Table 1, if considered desirable for use, may be tested similarly.

EXAMPLE No. 7A



Axial compression member — allowable load, grade C steel (using Chart 3B, p. 8 and Chart 4, p. 11).

STEEL = Grade "C": $f_b = 18{,}000$ p.s.i.; $f_y = 33{,}000$ p.s.i.
LENGTH L = 10 feet
TO FIND: Allowable Axial Load, P, for the section indicated (Sec. 3.6, Design Specifications, in Part III).

Sectional properties are computed on the basis of full section:

$A = 0.636 \text{ in.}^2$

$r_x = 1.02$ in.

The value of Q (Sec. 3.6.1, Design Specifications, in Part III) is determined as follows:

$$\frac{w_1}{t} = \frac{2.76}{0.06} = 46.0; \frac{w_2}{t} = \frac{2.26}{0.06} = 37.7$$

Effective design widths for $f_b = 18{,}000$ p.s.i. according to Chart 3 B, p. 8, ar

$$b_1 = 35.4t = 2.12 \text{ in.}; b_2 = 31.3t = 1.88 \text{ in.}$$

Then, $A_{eff} = 0.636—0.06[2(2.76—2.12) + 2(2.26—1.88)] = 0.514$ in.^{2}
Since the member is composed entirely of stiffened elements.

$$Q_A = \frac{\text{Effective Area}}{\text{Gross Area}} = \frac{0.514}{0.636} = 0.807$$

$$\text{Slenderness ratio} = \frac{L}{r_x} = \frac{10(12)}{1.02} = 118$$

From column design curves of Chart 4, p. 11, for $Q = 0.807$ and $\frac{L}{r_x} = 118$

$$\frac{P}{A} = 8300 \text{ p.s.i. acting on full area of section}$$

$$P = 8300\ (0.636) = 5270 \text{ lb.}$$

EXAMPLE No. 7B

Axial compression member — allowable load, grade A steel (using Chart 3B, p. 8 and Chart 4, p. 11).

STEEL = Grade "A": $f_b = 13{,}500$ p.s.i.; $f_y = 25{,}000$ p.s.i.
LENGTH L = 10 feet
TO FIND: Allowable Axial Load, P. (Sec. 3.6, Design Specification, in Part III)
SECTION: Same as Example 7a.

The full sectional properties and flat-width ratios are:

$$A = 0.636 \text{ in.}^2;\ r_x = 1.02 \text{ in.};\ \frac{w_1}{t} = 46.0;\ \frac{w_2}{t} = 37.7$$

For $f_b = 13{,}500$ p.s.i. the effective design widths, from Chart 3 B, p. 8, for the w/t ratios indicated above, are:

$b_1 = 37.1t = 2.23$ in.; $b_2 = 32.3t = 1.94$ in.

Thus the effective area:

$$A_{eff} = 0.636—0.06[2(2.76—2.23) + 2(2.26—1.94)] = 0.534 \text{ in.}^2$$

Since the member is composed entirely of stiffened elements,

$$Q_A = \frac{\text{Effective Area}}{\text{Gross Area}} = \frac{0.534}{0.636} = 0.839$$

$$\text{Slenderness Ratio, } \frac{L}{r_x} = \frac{10(12)}{1.02} = 118$$

To use for Grade A steel the column design curves of Chart 4, (Grade "C" Steel), enter the chart with:

$$Q = Q_A\left(\frac{f_y}{33000}\right) = 0.839\left(\frac{25000}{33000}\right) = 0.63, \text{ and } \frac{L}{r_x} = 118 \text{ to get:}$$

$$\frac{P}{A} = 7200 \text{ p.s.i. acting on full area of section}$$

$$P = 7200\ (0.636) = 4575 \text{ lb.}$$

EXAMPLE No. 7C

Axial compression member — allowable load, steel 50,000 yield point, (using Chart 4, p. 11 and Chart 3A, p. 7 or Chart 3B, p. 8).

STEEL = 50,000 p.s.i. yield point

LENGTH L = 10 feet

TO FIND: Allowable Axial Load, P (Sec. 3.6, Design Specification, in Part III)

SECTION: Same as Example No. 7a.

The full sectional properties and flat-width ratios are:

$$A = 0.636 \text{ in.}^2;\ r_x = 1.02 \text{ in.};\ \frac{w_1}{t} = 46.0;\ \frac{w_2}{t} = 37.7$$

From Specification, Sec. 3.1 $f_b = \dfrac{50{,}000}{1.85} = 27{,}000$ p.s.i.

The value of Q is determined by obtaining from Chart 3A, p. 7, or Chart 3B, p. 8, the effective widths, b, for $f_b = 27{,}000$, and the w/t ratios indicated above, as follows:

$b_1 = 32.2t = 1.93$ in.; $b_2 = 29.1t = 1.75$ in.

Thus the effective area:

$$A_{eff} = 0.636—0.06[2(2.76—1.93) + 2(2.26—1.75)] = 0.475 \text{ in.}^2$$

Since the member is composed entirely of stiffened elements,

$$Q_A = \frac{\text{Effective Area}}{\text{Gross Area}} = \frac{0.475}{0.636} = 0.746$$

$$\text{Slenderness ratio, } \frac{L}{r_x} = \frac{10(12)}{1.02} = 118$$

To use for 50,000 p.s.i. yield point steel the column design curves of Chart 4, (Grade "C" steel), enter the chart with:

$$Q = Q_A \left[\frac{f_y}{33000} \right] = 0.746 \left[\frac{50000}{33000} \right] = 1.13, \text{ and } \frac{L}{r_x} = 118 \text{ to get:}$$

$$\frac{P}{A} = 9400 \text{ p.s.i. acting on full area of section}$$

$$P = 9400\ (0.636) = 5970 \text{ lb.}$$

EXAMPLE No. 8

Column with combined axial and bending stresses (using Table 2).

REQUIRED: *A column (Table 2) or primary member adequate to support* axial load and bending, for the basic $f_b = 18,000$ p.s.i. (Webs of component channels are connected by closely spaced welds that insure action of the two webs together, not individually, i.e. Q_{max} applies.)

STEEL: Grade C ($f_b = 18,000$ p.s.i.)

LENGTH: L = 8 ft. 4 in., = 100 in. (Adequately braced about Y-Y axis, unbraced about X-X axis.)

AXIAL LOAD: P = 10,000 lb.

APPLIED MOMENT about X-X axis: $M_x = 25,000$ in.-lb.

For a column or primary support, L/r must not exceed 120, (Sec. 3.6.2, Design Specification, in Part III). The column is adequately braced about the Y-Y axis, consequently r_x will govern and must be at least 100/120 or 0.833 in. The section must therefore be selected from among those sections which have r_x values greater than 0.833 in.

If bending stresses alone were involved, the required section modulus, S_x, would be $\frac{25,000}{18,000}$ or 1.39 in.^{3}. For *initial trial selection*, a section having about twice the required section modulus is taken: 2 x 1.39 in.^{3} = 2.78 in.^{3}. The section whose section modulus, S'_x, most nearly approximates this value is the section of nominal dimensions 6″ x 3″ No. 12 ga., for which $S'_x = 2.82$ in.^{3}; $Q = 0.995$; $r_x = 2.17$; $K = 1.0$; area = 1.80 sq. in. Testing this section (Sec. 3.7, Design Specification, in Part III):

$$\begin{aligned} \text{Allowable axial unit stress, } F_a &= 15{,}300 \times Q - 0.437\ Q^2\ (L/r_x)^2 \\ &= 15{,}300 \times 0.995 - 0.437 \times 0.995^2\ (100/2.17)^2 \\ &= 15{,}224 - 919 \\ &= 14{,}305 \text{ p.s.i.} \end{aligned}$$

Allowable bending unit stress in compression:

$$\begin{aligned} F_b &= f_b\ K \\ &= 18{,}000 \times 1.00 \\ &= 18{,}000 \text{ p.s.i.} \end{aligned}$$

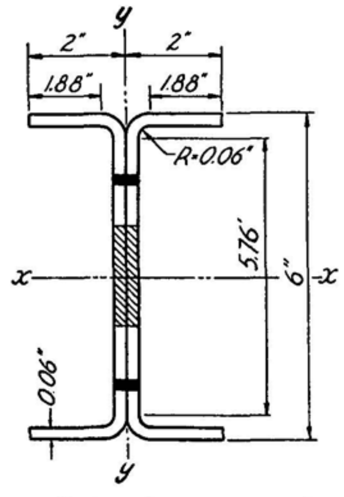
$$\text{Axial unit stress, } f_a = \frac{\text{axial load}}{\text{Area of Section}} = P/A$$
$$= \frac{10{,}000}{1.80}$$
$$= 5556 \text{ p.s.i.}$$

$$\text{Bending unit stress, } f'_b = \text{Applied moment} \div \frac{S'_x}{K}$$
$$= 25{,}000 \div \frac{(2.82)}{(1.00)}$$
$$= 8865$$

$$\left(\frac{f_a}{F_a} + \frac{f'_b}{F_b}\right) = \left(\frac{5556}{14{,}305} + \frac{8865}{18{,}000}\right) = 0.388 + 0.493 = 0.881$$

Since 0.881 does not exceed unity (Sec. 3.7, Design Specification, in Part III), the section tentatively selected is suitable for use. Other sections near this section in Table 2 may be tested similarly. In this example, the first section selected for investigation was the lightest section that appeared to qualify.

EXAMPLE No. 9



Wall stud braced by wall-sheathing — axial compression member (stiffened and unstiffened elements).

STEEL: Grade "C"

LENGTH L = 15 feet

Stud faced on both flanges by wall-sheathing of adequate strength.

TO FIND: Allowable design load and maximum spacing of wall-sheathing attachments.

In accordance with Sec. 3.6, Design Specification, in Part III, the cross-sectional properties* are to be computed on the basis of the full section, hence

* In this example the properties were calculated for the actual cross-section (with rounded corners), not with square corners as was assumed in the preceding examples to simplify the calculations.

$A = 1.159$ in.^{2} $\quad$ $I_y = 0.641$ in.^{4}
$I_x = 6.04$ in.^{4} $\quad$ $r_y = 0.74$ in.
$r_x = 2.28$ in.

The value of Q (Sec. 3.6.1, Design Specification, in Part III) is determined as follows:

The member is composed of both stiffened and unstiffened elements. Paragraph (c) of Sec. 3.6.1 therefore applies.

For the flanges, $w/t = \dfrac{1.88}{0.06} = 31.3$

Since the flanges are unstiffened elements,
from Sec. 3.2: $f_c = 12600 - 148.5(31.3) = 7950$ lb. per sq. in.

whence $\quad Q_s = \dfrac{7950}{18000} = 0.44$

For the webs (stiffened elements):

$$w/t = \frac{5.76}{0.06} = 96^*$$

b/t (Sec. 2.3.1, Design Specification, in Part III, and Chart No. 3A, p. 7), based on f = 7950 lb. per sq. in., equals 62. Since the full area of all unstiffened elements (as well as the effective [reduced] area of all stiffened elements) is to be included in computing Q_a (Sec. 3.6.1, Design Specification, in Part III) the reduction of web (stiffened) section is first calculated:

$w - b = (96 - 62)\ t = (34)\ (0.06) = 2.04$ in.

The corresponding area reduction (for the two webs) is:

$2.04 \times 0.06 \times 2 = 0.245$ in.^{2}

The net area of the stud is:

$1.159 - 0.245 = 0.914$ in.^{2}

Thus, $Q_a = \dfrac{0.914}{1.159} = 0.79$

Then $Q = Q_s \times Q_a = (0.44)(0.79) = 0.35$

Allowable spacing of wall-sheathing attachments (for k_{min}) from Sec. 5:

$$a = r_y\sqrt{\frac{29{,}500{,}000}{33{,}000}} = 30r_y = 30(0.74) = 22.2 \text{ in.}$$

Chosen spacing: a = 22.0 in. to afford lateral bracing in plane of wall.

Slenderness ratio, (perpendicular to wall): $\dfrac{L}{r_x} = \dfrac{15\ (12)}{2.28} = 78.9$

From the column design curves in Chart 4, p. 11, for

$Q = 0.35$ and $\dfrac{L}{r} = 78.9$

$P/A = 5050$ p.s.i. acting on full area of section

$P = 5050\ (1.159) = 5850$ lb.

* If the weld spacing is such that the two web elements may be considered acting as one, the w/t value may be taken as 48.

To determine the properties of the wall materials and attachments required to adequately brace the studs laterally, refer again to Sec. 5, Design and Specification, in Part III. From paragraph (c) of that section,

$$k = \frac{4.5a\ A^2}{I_z} = \frac{4.5 \times 22 \times \overline{1.159}^2}{0.641} = 207$$

From paragraph (d) of Sec. 5,

$$F_{min} = \frac{k\ e\ P}{2\sqrt{E\ I_2\ k/_a} - P} = \frac{207 \times 0.75 \times 5850}{2\sqrt{29{,}500{,}000 \times 0.641 \times 207/22} - 5850}$$

$$= 43.6 \text{ lbs.}$$

USE OF TABLES FOR OTHER GRADES OF STEEL

(See Comments, page 12)

The beam strength properties, S_x and S_y, are in many cases identical in value for the two grades of steel covered. For those sections these same values apply for any grade of steel. Wherever there is a difference between the value tabulated under $f_b = 18{,}000$ p.s.i., and the corresponding value tabulated under $f_b = 27{,}000$ p.s.i., a value for any other grade of steel can be obtained by straight-line interpolation or extrapolation based on the two tabulated values. So long as the extrapolation is not extended to values of f_b lower than 13,500 p.s.i. (Grade A steel) or above 30,000 p.s.i. (steel with a specified minimum yield of 55,500 p.s.i.) the interpolated or extrapolated values will be satisfactorily accurate. (See Comments, page 12.)

EXAMPLE No. 10a: (Table 1, Double Channels with Stiffened Flanges)

Steel: Grade A ($f_b = 13{,}500$ p.s.i.)
Section from Table 1, nominal dimensions 7″ x 5½″ x No. 16 ga.
To find: S_x
From Table 1: $S_x = 3.16$ ($f_b = 18{,}000$) and 3.06 ($f_b = 27{,}000$).
Hence, from p. 12:

$$S_x \ (\text{for } f_b = 13{,}500) = 3.16 + \left[(3.06 - 3.16)\left(\frac{13{,}500 - 18{,}000}{27{,}000 - 18{,}000}\right)\right]$$
$$= 3.16 + 0.05$$
$$= 3.21 \text{ in.}^3$$

The deflection properties, I_x and I_y, do not vary with the grade of steel used and therefore no interpolation or extrapolation is required.

The column properties Q, Q_{min} and Q_{max} are determined in exactly the same manner as the beam strength properties, as explained above, and the limits of extrapolation for satisfactory accuracy are the same.

EXAMPLE No. 10b: (Table 4, Zee with Stiffened Flanges)

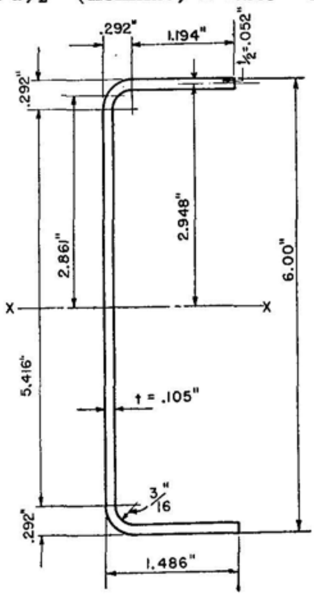
STEEL: Specified minimum yield 40,000 p.s.i.
($f_b = 21{,}622$ p.s.i.)
Zee section from Table 4, nominal dimensions
9″ x 3¼″ x No. 14 ga.
To find: Q
From Table 4, Q = 0.629 ($f_b = 18{,}000$) and 0.574 ($f_b = 27{,}000$)
Hence, interpolating by the procedure discussed on p. 12:

$$Q \text{ (for } f_b = 21{,}622 \text{ p.s.i.)} = 0.629 + \left[(0.574-0.629)\left(\frac{21{,}622-18{,}000}{27{,}000-18{,}000}\right)\right]$$
$$= 0.629 - 0.022$$
$$= 0.607$$

Linear Method.

STEEL: Any Grade.
TO FIND: Strength in Bending about Major Axis.
SECTION: Standard 6″ x 1½″ (nominal) x 0.105″ Channel with Unstiffened Flanges.

EXAMPLE No. 11



Properties of 90° Corner:
Rad. Inside: 0.1875″
Rad. Outside: 0.1875″ + 0.105 = 0.2925″, say 0.292″

Mean Radius: $R = \dfrac{0.2925 + 0.1875}{2} = 0.240$

Length of Arc, $L = 1.57 \times 0.240 = 0.377''$ (See p. 41)
I′ of corner about its centroidal axis is negligible.

Distance of C. G.: $c = 0.637 \times 0.240 = 0.153''$
Flat Width of Flange: $1.486 - 0.292 = 1.194''$
Flat Width of Web: $6.00 - (2 \times 0.292) = 5.416''$

Distance from X-X Axis to Center of Flange: $3.00 - \dfrac{0.105}{2} = 2.948''$

Distance from X-X Axis to C. G. of Corner: $\dfrac{5.416}{2} + 0.153 = 2.861''$

Since w/t of flange $= \dfrac{1.194}{0.105} = 11.37$, less than 12, full section properties apply.

COMPUTATION of Linear I'_x

WEB:	$1/12 \times 5.416^3$	$= 13.24$
CORNERS:	$2 \times 0.377 \times 2.861^2$	$= 6.17$
FLANGES:	$2 \times 1.194 \times 2.948^2$	$= 20.75$
I'_x		$= 40.16$ in.³

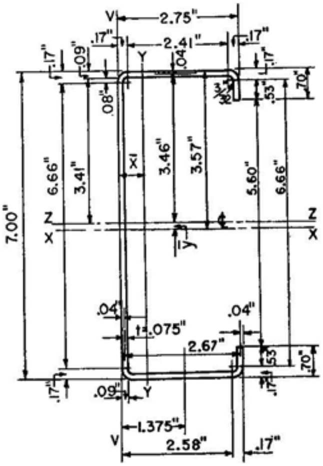
Actual $I_x = I'_x \times t = 40.16 \times 0.105 = 4.22$ in.^{4} (See p. 40)

$$S_x = \frac{4.22}{3} = 1.41 \text{ in.}^3$$

These properties are identical to those in Table 5 of the Manual.

Linear Method.

EXAMPLE No. 12



STEEL: Grade C.

TO FIND: (a) Column Properties.
(b) Strength in Bending about Major Axis.

SECTION: Standard 7″ x 2¾″ x 0.075″ Channel with Stiffened Flanges.

Properties of 90° Corner:

Rad. Inside = 0.09375″, say 0.094
Rad. Outside = 0.09375 + 0.075 = 0.16875, say 0.17.
Mean Radius, R = ½ (0.094 + 0.17) = 0.13″
Length of Arc, L = 1.57 × 0.13 = 0.206, say 0.21″
Distance to C. G. = 0.637 × 0.13 = 0.084, say 0.08″
I′ of corner about its centroidal axis is negligible.

Compute other dimensions and distances from Z-Z axis (center line of section) and V-V axis to center of gravity of elements as shown.

a. COLUMN PROPERTIES:

1. Computation of I'_z

WEB	$1/12 \times 6.66^3$	=	24.62
STRAIGHT PARTS OF LIPS	$1/12\ (6.66^3 - 5.60^3)$	=	9.98
4 CORNERS	$4 \times 0.21 \times 3.41^2$	=	9.77
FLANGES	$2 \times 2.41 \times 3.46^2$	=	57.70
	For Full Section ($\bar{y} = 0$): $I'_z: = I'_x:$	=	102.07 in.^{3}

$I_z = I'_z \times t = 102.07 \times 0.075 = 7.655$ in.^{4} (Table 7, conventional $I_x = 7.660$ in.^{4}, full section).

2. Computation of I'_v. Distance from V-axis: $\bar{x}$; statical moment of element: $1 \cdot \bar{x}$

Element	Length L (in.)	$\bar{x}$ (in.)	$L \cdot \bar{x}$ (in.^{2})	$L \cdot \bar{x}^2$ (in.^{3})	I' about c. g. of elements
WEB	6.66	0.04	0.27	0.01	0
STRAIGHT PART OF LIPS	1.06	2.71	2.87	7.78	0
2 NEAR CORNERS	0.42	0.09	0.04	0	0
2 FAR CORNERS	0.42	2.66	1.12	2.98	0
FLANGES	4.82	1.375	6.63	9.12	$2 \times 1/12 \times 2.41^3 = 2.34$ in.^{3}
Total length L_t =	13.38	Summation	10.93	19.89	2.34 in.^{3}

$$\bar{x} = \frac{10.93}{13.38} = 0.817'' \text{ (Table 7, x = 0.8150'').}$$

$I'_v = 19.89 + 2.34 = 22.23$ in.^{3}
$I'_y = I'_v - L_t (x)^2 = 22.23 - (13.38 \times 0.817^2) = 13.31$ in.^{3}
$I_y = I'_y \times t = 13.31 \times 0.075 = 0.998$ in.^{4} (Table 7, $I_y = 1.00$ in.^{4}).

3. Full Section Properties:

$A = L_t \times t = 13.38 \times 0.075 = 1.003$ in.^{2} (By Table 7, A = 1.003 in.^{2}).

$$r_x = \sqrt{\frac{I'_x}{L_t}} = \sqrt{\frac{I'_z}{L_t}} = \sqrt{\frac{102.07}{13.38}} = 2.76 \text{ in. (Table 7, } r_x = 2.76 \text{ in.)}$$

$$r_y = \sqrt{\frac{I'_y}{L_t}} = \sqrt{\frac{13.31}{13.38}} = 0.997 \text{ in. (Table 7, } r_y = 0.999 \text{ in.)}$$

4. Computation of Q (Sec. 3.6, Design Specification)

FLANGES: $w/t = \dfrac{2.41}{0.075} = 32.1$, $b/t = 28.5$;

Deduction: $2(32.1-28.5) \times 0.075 = 0.54''$

WEB: $w/t = \dfrac{6.66}{0.075} = 88.8$, $b/t = 45.6$;

Deduction: $(88.8-45.6) \times 0.075 = 3.24''$

TOTAL EFFECTIVE LENGTH: $L_{eff} = 13.38-0.54-3.24 = 9.60''$

$$Q = \frac{L_{eff}}{L_t} = \frac{9.60}{13.38} = 0.718 \text{ (Table 4, Q = 0.718)}$$

(b) STRENGTH IN BENDING ABOUT MAJOR AXIS:

The deduction from the flat width of the compression flange is $(32.1 - 28.5) \times 0.075 = 0.27''$ (see above).

The reduced I'_z is then: $102.07 - (0.27 \times 3.46^2) = 98.84$ in.^{3}.

Total effective length is in this case: $L_{eff} = 13.38 - 0.27 = 13.11''$.

The location of the X-X axis through the center of gravity of the section with this effective length, and in the distance $\bar{y}$ from the Z-Z axis, is found from the equation: $13.11 (\bar{y}) = 0.27 \times 3.46 = 0.935$; $\bar{y} = 0.07''$.

We now have $I'_x = I'_z - (13.11 \times 0.07^2) = 98.84 - 0.06 = 98.78$ in.^{3}.

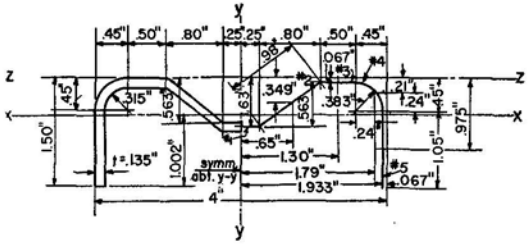
Distance from X-X axis to the extreme fiber is: $3.50 + 0.07 = 3.57''$

$$S_x = \frac{I'_x \times t}{3.57} = \frac{98.78 \times 0.075}{3.57} = 2.075 \text{ in.}^3 \text{ (Table 4, } S_x = 2.08 \text{ in.}^3\text{)}$$

$$M_{max} = f_b \times S_x = 18{,}000 \times 2.075 \text{ in.}^3 = 37{,}350 \text{ in.-lb.}$$

Linear Method.

EXAMPLE No. 13



STEEL: Any Grade.

TO FIND: a. Column Properties.
b. Strength in Bending about the Minor Axis.

NOTE: That the web element of this section is effectively stiffened by the longitudinal corrugation. The small radii of the sharp bends on the web permit the web to be considered as composed of five straight elements without connecting curves. The web-to-flange bends are of greater radius and are computed as 90° corners.

CORNER ELEMENT NO. 4:

Mean Rad. $= 0.315 + 0.068 = 0.383''$

$L = 1.57 \times 0.383 = 0.60''$

$c = 0.637 \times 0.383 = 0.24''$; hence $\bar{y} = 0.45'' - 0.24'' = 0.21''$

$I' = 0.149 \times 0.383^3 = 0.008 \text{ in.}^3$

Compute I'_z for the half section; $\frac{1}{2}I'_z = \frac{1}{2}I_z/t$

Element	Length L (in.)	Distance from Z-axis: $\bar{y}$ (in.)	$L \cdot \bar{y}$ (in.^{2})	$L \cdot \bar{y}^2$ (in.^{3})	I′ of elements about their own axes:
#1	0.25	0.63	0.157	0.099	0
#2	0.98	0.349	0.342	0.119	$\frac{0.98 \times 0.563^2}{12} = 0.026$
#3	0.50	0.067	0.033	0.002	0
#4	0.60	0.21	0.126	0.026	See above $= 0.008$
#5	1.05	0.975	1.024	0.998	$\frac{1.05^3}{12} = 0.096$

$\frac{1}{2} L_t = 3.38$ Summation: 1.682 1.244 0.130

$\frac{1}{2} I'_z = 1.244 + 0.130 = 1.374 \text{ in.}^3$

$$\bar{y} = \frac{1.682}{3.38} = 0.498''$$

Compute I'_x for the half section:

$$\tfrac{1}{2} I'_x = \tfrac{1}{2} I'_z - \tfrac{1}{2} L_t (\bar{y})^2 = 1.374 - (3.38 \times 0.498^2) = 0.536 \text{ in.}^3$$

$$r_x = \sqrt{\frac{2 \times 0.536}{2 \times 3.38}} = 0.40 \text{ in.}$$

AREA $= 2 \times 3.38 \times 0.135 = 0.91$ in.^{2}

DISTANCE FROM C. G. TO EXTREME FIBER: $1.50 - \bar{y} = 1.50 - 0.498 = 1.002''$

$$S_x = \frac{2 \times 0.536 \times 0.135}{1.002} = 0.145 \text{ in.}^3$$

$M_{max} = f_b \cdot S_x = 0.145\, f_b$ in-lb

Compute I'_y for the half section:

Element		I′
#1	$1/3 \times 0.25^3$	$= 0.005$
#2	$0.98\left(0.65^2 + \frac{0.80^2}{12}\right)$	$= 0.466$
#3	$0.50\left(1.30^2 + \frac{0.50^2}{12}\right)$	$= 0.855$
#4	$0.60 \times 1.79^2 + 0.008$	$= 1.930$
#5	1.05×1.933^2	$= 3.923$

Summation: $1/2\, I'_y = 7.179$ in.^{3}

$I'_y = 2(7.179) = 14.358$ in.^{3}

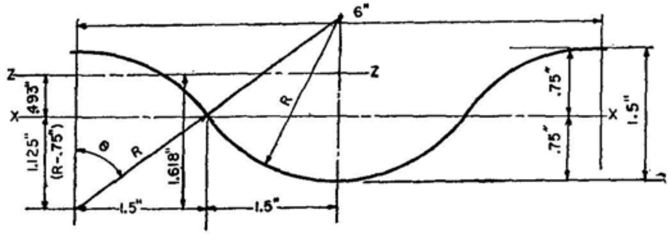
$I_y = I'_y \cdot t = 14.358(0.135) = 1.938$ in.^{4}

$$S_y = \frac{14.358 \times 0.135}{2.00} = 0.969 \text{ in.}^3$$

$$r_y = \sqrt{\frac{2(7.179)}{2(3.38)}} = 1.46 \text{ in.}$$

Linear Method.

EXAMPLE No. 14



STEEL: Any Grade.

TO FIND: Moment of Inertia of the section about the minor axis.

NOTE: While corrugated sheet sections are not specifically covered by the Specification, the computation of their section properties is handled most expeditiously by the linear method.

From the figure we have:

RADIUS R: $R^2 = (R-0.75)^2 + 1.5^2$, which gives $R = 1.875''$.

$$\text{ANGLE } \theta: \quad \sin\theta = \frac{1.5}{1.875} = 0.8: \quad \cos\theta = \frac{1.125}{1.875} = 0.6$$

$$\theta = 53°8' = 0.927 \text{ radians.}$$

The distance of the center of gravity of the arc with central angle θ from the arc center is $c_1 = \dfrac{R \sin \theta}{\theta} = 1.618''$. (See p. 41.)

Length of Arc: $L = R\,\theta = 1.74''$. For this arc we now find I'_z for the Z-Z axis thru the center of gravity for each half of the section above and below the center axis X-X from:

$$I'_z = R^3 \left(\frac{\theta + \sin\theta\cos\theta}{2} - \frac{\sin^2\theta}{\theta}\right) =$$

$$1.875^3 \left(\frac{0.927 + (0.8 \times 0.6)}{2} - \frac{0.8^2}{0.927}\right) = 0.086 \text{ in.}^3$$

The distance of the Z-Z axis from the X-X axis is: $1.618 - 1.125 = 0.493''$ (see figure).

For the arc with central angle θ we have then: $I'_x = 0.086 + (1.74 \times 0.493^2) = 0.509 \text{ in.}^3$

Since the horizontal projection of this arc is 1.5″, the actual I_x of the section per 12″ width for a thickness, t, is $I_x = 8 \times 0.509 \times t = 4.07 \times t$ (in.^{4}).

Area per 12″ width: $A = 8 \times L\,t = 8 \times 1.74 \times t = 13.9 \times t$ (in.^{2}).

APPENDIX — SUPPLEMENTARY TABLES

The conventional section or structural *properties based on the full cross-section* of light gage cold-formed steel structural members *are not generally proper criteria of their strength or elastic behavior.* However, the computation of design properties of these sections, as required by the Design Specification, occasionally involves the determination of some of these conventional section properties as a preliminary to the computations stipulated by the Specification.

When calculating the proper design properties and dimensions for the various sections that appear in the Tables in Part II, the conventional structural properties of the full (unreduced) cross-sections also were calculated. These conventional properties are presented in the tables that follow, together with certain additional data.

Detailed dimensions and properties of the separate component elements of the standard sections also are included in the tables that follow. These data should be helpful in the design analysis of any sections which are composed, even in part, of the elements included in the tables that follow.

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